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COMPARATIVE TESTS ON EXPERIMENTAL DRAFT-TUBES

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To Be Presented January 17, 1924

Synopsis

This paper gives the results of tests, made at the Alden Hydraulic Laboratory, on twelve model draft-tubes, to determine the relative efficiencies of draft-tubes of different types under the hydraulic conditions existing at a 120 000-h. p. hydro-electric power plant now under construction.

The data and results are by no means considered final and comprehensive. They constitute a study of one specific problem, and are given to show that a great amount of research work, of which this is only a part, must be done before a complete set of empirical rules will be available for use in draft-tube design. It is hoped that the publication of this information will be the means of stimulating further and more valuable investigation of this important subject.

The accuracy and value of tests of this nature have often been questioned, sometimes because of a lack of knowledge of the conditions under which the tests were made; for this reason, this paper gives a detailed description of the apparatus and methods used in making these tests. Precautions were taken for eliminating the usual sources of error, and it is believed that the relative values, and perhaps the absolute values, of efficiencies here shown, are accurate to a very high degree.

Purpose of the Tests

The proper design of draft-tubes for hydro-electric plants had received very little attention from hydraulicians, until, following a period of very bad design which culminated just prior to the World War, there came a demand for better efficiencies. When the Alabama Power Company in 1921 undertook to design and build the new hydro-electric plant known as Mitchell Dam, the matter of proper design of draft-tubes for a plant of this type had already received considerable attention from engineers and hydraulic turbine manufacturers, and there was in evidence a tendency to discard completely the elbow or quarter-turn tube, which, for a number of years, had been the conventional type of design. Several new and highly efficient tubes designed on basic hydraulic

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in April, 1924, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

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principles had appeared, and the test data and theory on which they were based, together with the actual operating performance at a number of newly completed plants, indicated that the designers had valid claims for efficiencies much higher than could be obtained from tubes of the quarter-turn type. Complete tests of the various types, however, had not been made under identical conditions, and it was impossible for the prospective user to determine from available published data either the relative merits of the types of tubes offered or the type best suited to his needs. Some of the tests, furthermore, were made with the draft-tube acting alone; and, although they were of great value to the draft-tube designer, the performance under this condition obviously did not apply when the tube was operating in connection with a water-wheel.

Consequently, the Alabama Power Company authorized a series of tests on draft-tube models to be made at the Alden Hydraulic Laboratory of the Worcester Polytechnic Institute, at Worcester, Mass., for the purpose of obtaining a direct comparison of the efficiencies of various types of draft-tubes and of determining their relative merits under conditions existing at the various power sites in which the Company is interested. The tests were made by the writers under the direction of O. G. Thurlow, Assoc. M. Am. Soc. C. E., Chief Engineer of the Alabama Power Company.

It was decided that a comparison of definite value could be obtained only by testing the models under identical conditions, and only by observing the over-all performance of the tube and water-wheel models acting as a unit. The arrangement of the testing laboratory and instruments was made with these points constantly in mind.

It was first planned to conduct tests on only five models; but other meritorious designs were submitted, and the work was extended to cover twelve distinct types of tubes. The test work was started in January, 1922, and tests of the first five tubes were completed in 3 months. Practically all this testing was done while the temperature was below the freezing point, and, as a doubt was expressed as to the accuracy of the leakage corrections, the work was discontinued for 6 weeks while the flume was being lined with sheet-copper, and other improvements were being made. Tests of the last seven tubes were completed during the first week in August, 1922, making a total of about 6 months of actual testing.

Description of Model Runner and Tubes

The writers were fortunate in securing for use in these tests a model runner built by the Allis-Chalmers Manufacturing Company and designed to be homologous with the 130-in., 24 000-h. p. runners furnished by that Company for Mitchell Dam. This model runner had been used in the Allis-Chalmers laboratory for determining the guaranteed performance of the Mitchell Dam water-wheels. It was a low-head, Francis type (Fig. 6), $10\frac{1}{2}$ in. in diameter, made of bronze, and had a specific speed of 66.7. The speeds attained with this runner varied from 300 to 600 rev. per min., with a maximum of 14 h. p.

Of the twelve experimental draft-tubes investigated in this series of tests, six were designed by water-wheel manufacturers, three by the Alabama Power Company, and three were designed and built in the laboratory by the writers.

The model tubes were designed for identical hydraulic conditions, and each was homologous with such a tube of its type as would fit the actual conditions at Mitchell Dam. Inasmuch as the tubes and runner were designed for the same set of specific conditions, the results of the tests may not be used for other conditions widely different; yet a direct comparison of the performance should afford a basis for general conclusions that would be of value in designing tubes for other conditions.

Table 1 is a list of the experimental tubes used in these tests, the designer or maker and type, and a reference to the diagram illustrating each tube. The dimensions of the experimental tubes, as related to those of full size, were in the ratio of 10½ to 130, which are the respective diameters, in inches, of the model runner and the Mitchell Dam runners. The tubes were made of pine, spruce, and oak. A fair idea of the method of construction may be obtained from the illustrations.

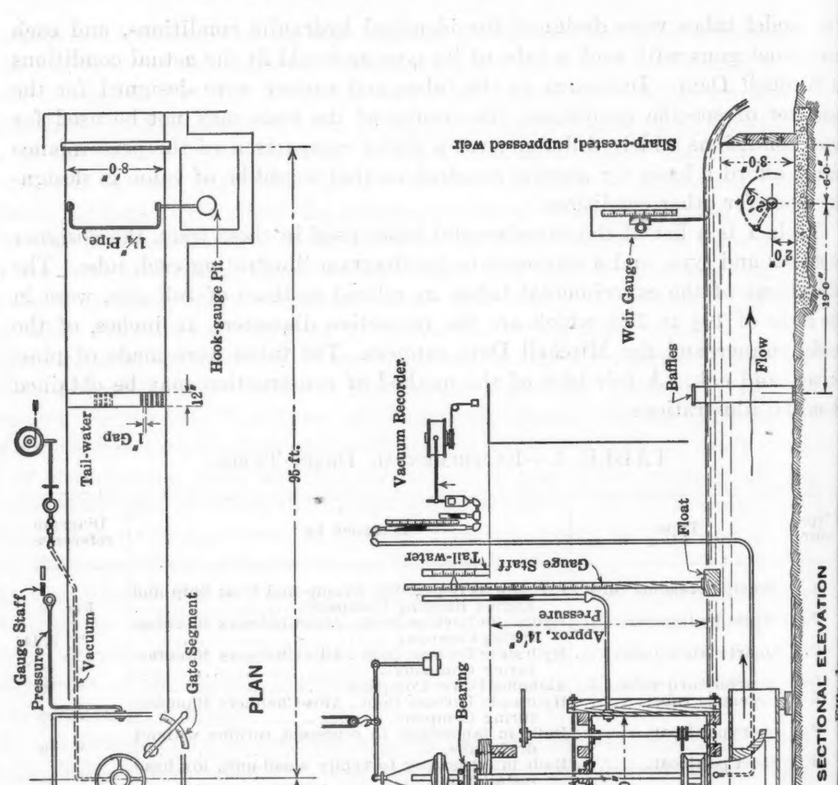
TABLE 1.—Experimental Draft-Tubes.

Type letter.	Type.	Designed by:	Diagram reference.
A	Moody spreading tube..	I. P. Morris Dept., Wm. Cramp and Sons Ship and Engine Building Company......................	Fig. 12
B	White hydraucone......	Hydraulic Turbine Dept., Allis-Chalmers Manufacturing Company..........................	" 9, 14
C	Quarter-turn tube......	Hydraulic Turbine Dept., Allis-Chalmers Manufacturing Company..........................	" 15
D	Quarter-turn tube......	Alabama Power Company......................	" 16
E	Eccentric tube..........	Hydraulic Turbine Dept., Allis-Chalmers Manufacturing Company..........................	" 17
F	Short petticoat..........	Built in laboratory to represent turbine without draft-tube..................................	" 19
G	Short petticoat..........	Built in laboratory to typify small-unit, low-head installation................................	" 20
H	Concentric tube.........	Hydraulic Turbine Dept., Wellman-Seaver-Morgan Company...............................	" 21
J	Eccentric tube..........	Hydraulic Turbine Dept., Wellman-Seaver-Morgan Company...............................	" 22
K	Moody spreading tube..	Modified design of Tube *A* (Fig. 12)................	" 23
L	Quarter-turn tube......	Alabama Power Company......................	" 24
M	Eccentric tube..........	Alabama Power Company......................	" 25

Description of the Plant and Testing Apparatus

Alden Hydraulic Laboratory.—The low-head testing plant of the Alden Hydraulic Laboratory was well equipped for making tests of this nature. It consisted essentially of a 6 by 8 by 10-ft. wooden pressure-box connected to the pond or forebay by a steel penstock, 46 in. in diameter and about 40 ft. long; a discharge canal, 8 ft. wide and about 95 ft. long, having a suppressed measuring weir, 3 ft. high, in the lower end; and the necessary appurtenances for controlling and gauging the water (Figs. 1 and 2). During the tests a maximum effective head of 14½ ft. and a maximum discharge of 14 cu. ft. per sec. were obtained.

The elevation of the pond was regulated by a capstan-operated head-gate at the outlet of the main supply reservoir. A finer adjustment of elevation was obtained with flash-boards conveniently placed near the testing apparatus, by which arrangement it was possible to maintain a constant head (to within 0.01 ft.) throughout a test. Water was admitted to the penstock through a



SKETCH OF TESTING PLANT
GENERAL LAYOUT
Not to Scale

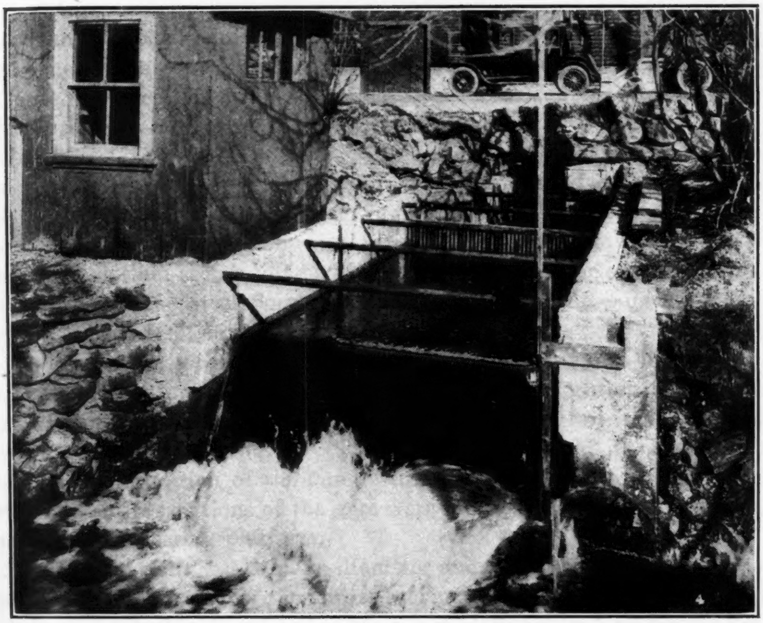


FIG. 2.—VIEW OF ALDEN HYDRAULIC LABORATORY, SHOWING SUPPRESSED WEIR, WITH HOOK-GAUGE HOUSE AT LEFT, AUXILIARY HOOK-GAUGE AT RIGHT, AND BAFFLES IN APPROACH CHANNEL.

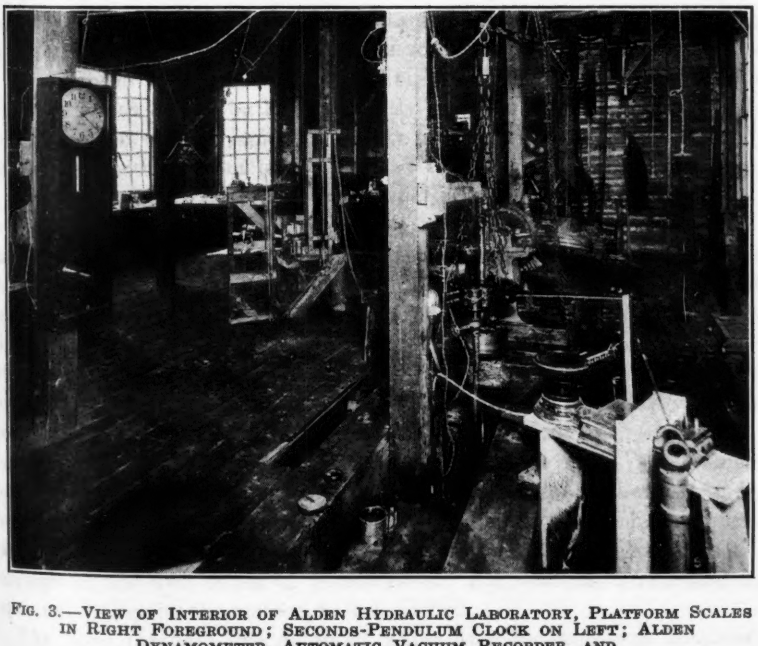


FIG. 3.—VIEW OF INTERIOR OF ALDEN HYDRAULIC LABORATORY, PLATFORM SCALES IN RIGHT FOREGROUND; SECONDS-PENDULUM CLOCK ON LEFT; ALDEN DYNAMOMETER, AUTOMATIC VACUUM RECORDER, AND HOOK-GAUGE IN CENTER.

Papers

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small hand-operated sluice-gate; when the pressure became equalized, the large penstock gate was opened. Suitable vents in the penstock and pressure-box were provided, in order to reduce the fluctuation in pressure due to a sudden opening and closing of the wicket gates.

The Wheel Setting.—The water-wheel casing was anchored with cap-screws to a ⅝-in. boiler-plate floor ring. This ring was leveled and attached to the bottom of the pressure-box with lag-screws. The model wheel-case was of cast iron in two pieces connected by steel pins which served as pivots for the wicket gates. The bottom section of the wheel-case supported the operating ring, and was secured to the steel floor plate. The upper section supported a water-lubricated, babbitted, steady bearing, about 4 in. long.

The wicket gates were of stream-line shape, made of cast iron. They were operated by a 1½-in. steel shaft extending through a tight stuffing-box in the top of the pressure-box, and provided with a lever arm for setting the gates at any desired opening. (Fig. 1.) The lever arm was held in place by a screw clamp. The calibration of the gate opening was made with small inside calipers, and definite positions of the gate were marked on a segment conveniently placed near the testing platform.

The turbine shaft was 3 in. in diameter and 11 ft. long, with the upper end tapered to 2 in. to receive the thrust-bearing and dynamometer. It was supported on the turbine foundation by a self-aligning, radial and thrust ball-bearing, running in oil. The thrust collar was clamped to the shaft with two heavy bolts through a split ring. The runner was keyed to the shaft and held in place by a ¾-in. machine bolt. The shaft passed out of the top of the pressure-box through a loose, adjustable stuffing-box. The main bearing was supported on heavy timbers extending across the main room of the laboratory.

An overhead I-beam trolley, carrying a differential chain block, traveled the entire width of the building, directly over the pressure-box and tail-race. This trolley was used to handle all heavy parts and also to support the equalizing beam of the dynamometer.

The Measuring Weir.—The water-wheel discharge passed through a rock-lined culvert, and was measured over an 8-ft., sharp-crested, suppressed weir, placed about 95 ft. down stream from the center line of the wheel (Fig. 2). The top of the weir was 3 ft. above the bottom of the approach channel, and was crested with an adjustable ⅛-in. brass plate. The body of the weir was of 4-in. yellow pine. The approach channel was of smooth concrete. A rough baffle grating, placed about 25 ft. up stream from the weir, eliminated cross-currents and eddies in the channel and provided a very quiet flow of water over the weir. A 1¼-in., vertically-perforated, galvanized pipe, extending horizontally across the channel, 8 ft. from the weir, served as a piezometer for reading the depth over the crest. This pipe was pivoted with a 2-ft. radius for checking the effect of the location of the piezometer. A flush-wall piezometer was placed just below the pipe, and was used to get a direct comparison of the readings of the two types of piezometers. All readings mentioned in this paper were taken by the method first described.

The Weir Gauge.—The depth of water over the weir was obtained from readings on a hook-gauge placed in a pit alongside the channel. To set the

hook-gauge at zero on the weir crest, an auxiliary hook-gauge was mounted about 20 in. down stream from the weir, with a hook-gauge pot connected by 1-in. rubber hose to the pot of the main gauge (Fig. 2). A 24-in. machinist's level (of aluminum) was placed on the weir crest and on the hook of the auxiliary gauge, which was adjusted until it was exactly level with the crest. The machinist's level was then turned, end for end, and the setting checked (Fig. 4). After the gauge reading had been taken, water was poured into one of the pots until the air had been expelled and a convenient level established in both pots. Water was then poured into the other pot to cause a reversed flow and to make sure that no air remained in the pipe. When the water was again stationary, both gauges were lowered into their respective pots and readings taken. The difference between the two readings of the auxiliary gauge was added to that of the main hook-gauge, and this gave the exact weir-crest zero reading.

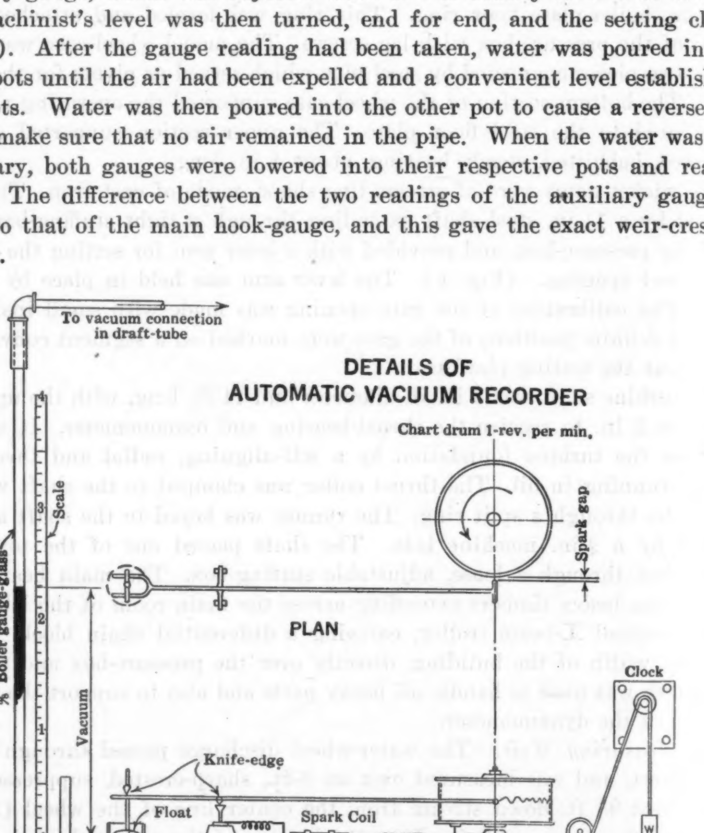

FIG. 4.

The Dynamometer.—The apparatus used in measuring the horse-power consisted of an Alden, 2-disk, 14-in., absorption dynamometer, with automatic valve; a seconds-pendulum clock; a solenoid revolution counter; an electric tachometer; a floor bell-crank; and platform scales with dash-pot (Fig. 5). The two cast-iron disks of the dynamometer were keyed to the shaft, and revolved in oil between thin copper plates. Water, circulating under pressure between the plates and casing, carried away the heat generated by fric-

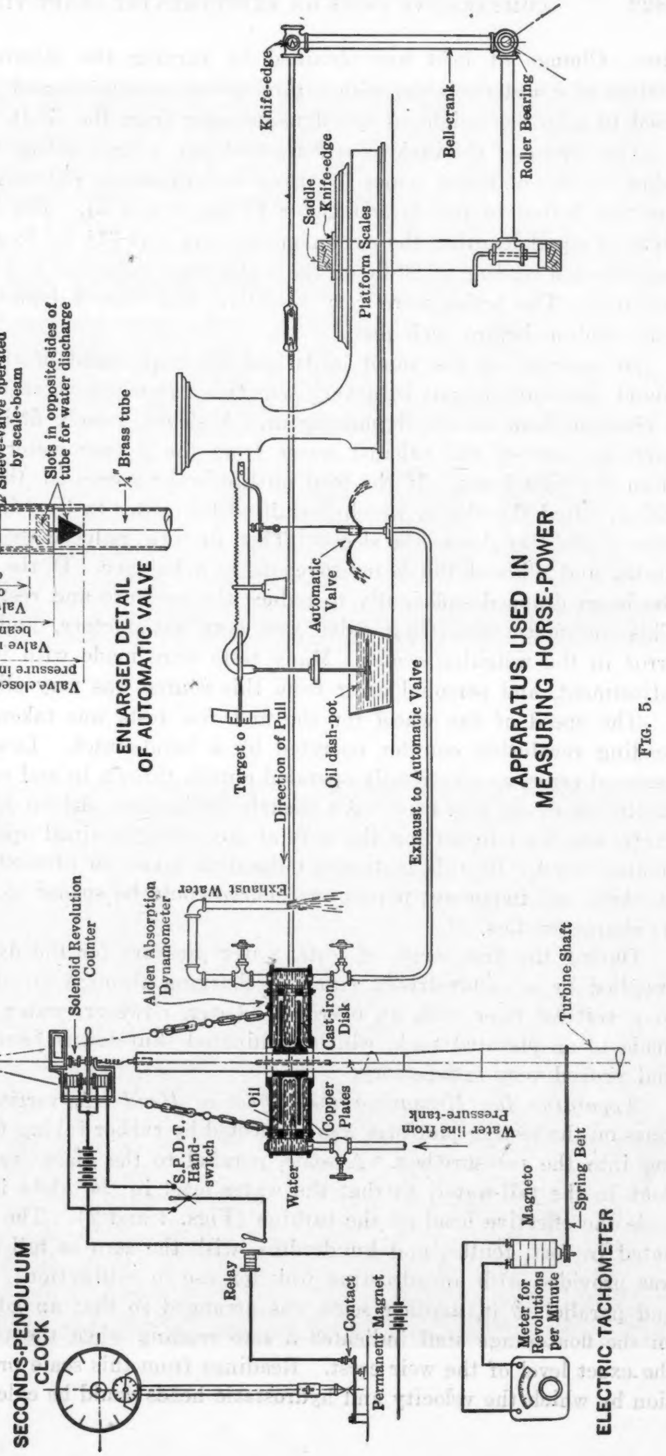
SECONDS-PENDULUM CLOCK
Permanent Magnet
Contact
Relay
S.P.D.T. Hand-switch
Solenoid Revolution Counter
Alden Absorption Dynamometer
Oil
Water
Copper Plates
Cast-iron Disk
Water line from Pressure-tank
Exhaust Water
Turbine Shaft
Spring Belt
Magneto
Meter for Revolutions per Minute
ELECTRIC TACHOMETER
Target
Direction of Pull
Oil dash-pot
Automatic Valve
Exhaust to Automatic Valve
ENLARGED DETAIL OF AUTOMATIC VALVE
Valve closed, increase of pressure in dynamometer
Valve normal, scale beam balanced
Valve opened, decrease of pressure in dynamometer
Sleeve-valve operated by scale-beam
Slots in opposite sides of tube for water discharge
1/4" Brass tube
Platform Scales
Saddle
Knife-edge
Bell-crank
Roller Bearing
APPARATUS USED IN MEASURING HORSE-POWER

FIG. 5.

tion. Change of load was obtained by varying the internal pressure. A system of counterweights, with a three-point suspension and ball thrust, was used to take the weight of the dynamometer from the shaft.

The torque of the brake was converted into a force acting vertically downward on the platform scales by using an adjustable rod and bell-crank and an arm bolted to the dynamometer (Figs. 3 and 5). The bell-crank arms were of equal lengths; the dynamometer arm was 31½ in. long. Under these conditions a reading of 20 lb. on the scales was equivalent to 1 h. p. at 100 rev. per min. The scales were very sensitive, and were balanced for an initial zero reading before each test.

On account of the small loads and the high speed of the wheel, it was found necessary to put in a very sensitive automatic valve (Fig. 5) to keep a constant load on the dynamometer. A sleeve, loosely fitting a small pipe carrying part of the exhaust water from the dynamometer, was suspended from the scale-beam. If the load on the brake increased, the scale-beam, in rising, lifted the sleeve, which permitted the water to flow from holes in the pipe at right angles to the sleeve. This, in turn, reduced the pressure on the brake, and allowed the beam to come to a balance. If the load decreased, the beam dropped sufficiently to reduce the pressure and restore the balance. This automatic throttling valve was very satisfactory, and introduced no error in the weighing scales. Many tests were made with it, without hand adjustment, and personal error from this source was very slight.

The speed of the wheel for the first few tests was taken with a direct-reading revolution counter operated by a hand-clutch. Later, to eliminate personal error, an electrically operated clutch, thrown in and out by a seconds-pendulum clock, was used. An electric tachometer, driven from the turbine shaft, was used in setting the gate at any predetermined opening for a particular speed. By this instrument the data taken on previous runs could be checked, and important points on a curve could be spaced so as to bring out its characteristics.

During the first series of tests, water pressure for the dynamometer was supplied by a motor-driven pump discharging through an air-pressure tank to a vertical riser with an overflow. Later, however, water connection was made to an elevated tank, which eliminated shut-downs from pump failures and proved very satisfactory.

Apparatus for Measuring the Effective Head.—A vertical 36-in. gauge glass on the testing platform was connected by rubber tubing to a pipe extending into the pressure-box. A scale, parallel to the glass, was attached to a float in the tail-water, so that the water level in the glass indicated on the scale the effective head on the turbine (Figs. 3 and 7). The scale was graduated in feet, tenths, and hundredths, with the zero at tail-water level, and was provided with an adjusting link for use in calibration. Near the glass, and parallel to it, another scale was arranged so that an adjustable pointer on the float-gauge staff indicated a zero reading when the tail-water was at the exact level of the weir crest. Readings from this scale provided information by which the velocity and hydrostatic heads could be calculated from the

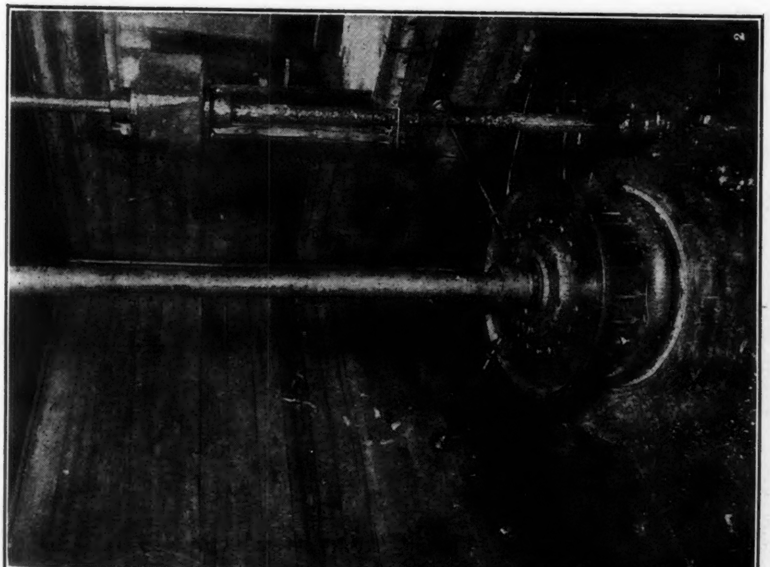

FIG. 6.—INTERIOR OF PRESSURE-BOX, ALDEN HYDRAULIC LABORATORY, WITH WATER-WHEEL IN POSITION.

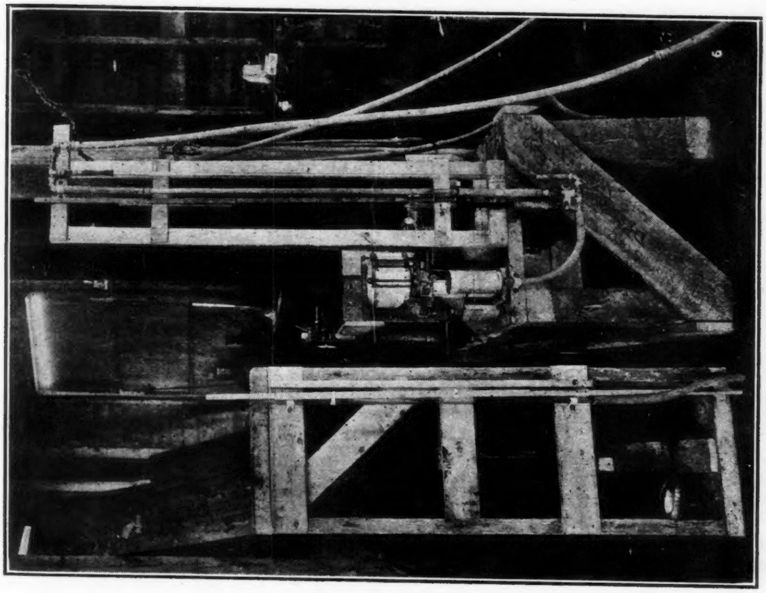

FIG. 7.—HEAD-GAUGE AND TAIL-WATER GAUGE STAFF AT LEFT; AUTOMATIC VACUUM RECORDING APPARATUS AT RIGHT, ALDEN HYDRAULIC LABORATORY.

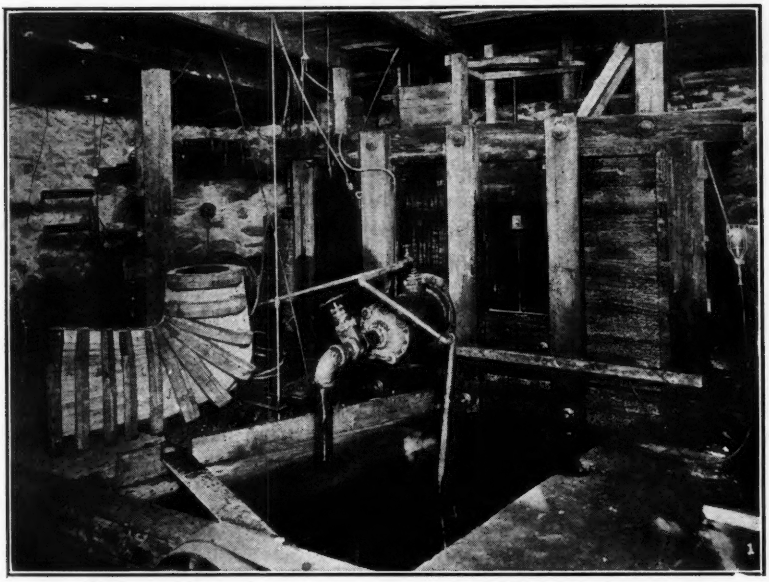

FIG. 8.—TAIL-RACE AND EXTERIOR OF PRESSURE-BOX, ALDEN HYDRAULIC LABORATORY; DRAFT-TUBE MODEL C, READY TO BE PLACED.

FIG. 9.—TYPE B DRAFT-TUBE IN PLACE READY FOR TESTING, ALDEN HYDRAULIC LABORATORY.

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vacuum readings. The float was of hard pine, 6 by 6 by 18 in., and was enclosed in a stilling basin formed by a removable wing-wall, 3 ft. deep and 10 ft. long, of dressed plank, and placed parallel to the line of flow in the tail-water culvert (Fig. 8).

Automatic Vacuum Recorder.—A vacuum recorder was devised for determining the draft head of the various tubes while under test (Fig. 10). This apparatus was essentially as follows: A vertical glass tube, 3 ft. long, closed at the upper end by the vacuum line from the draft-tube, was connected at the lower end by rubber hose to the bottom of a metal container filled with water, in which rested a copper float. A short glass tube, also in

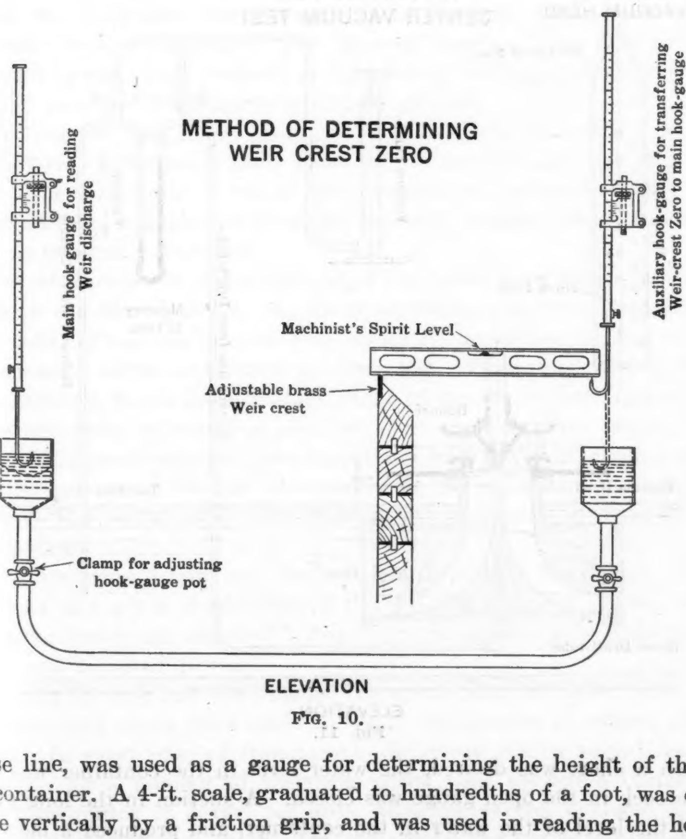


FIG. 10.

the hose line, was used as a gauge for determining the height of the water in the container. A 4-ft. scale, graduated to hundredths of a foot, was clamped in place vertically by a friction grip, and was used in reading the height of both water columns. A recording lever, resting on knife-edges, carried at one end the suspended copper float, also on knife-edges, and at the other end a recording metal point. This point cleared, by about $\frac{1}{32}$ in., the paper chart on a mechanically-operated chart drum. The recording lever was proportioned so that a fluctuation of 10 in. in the vacuum gauge registered as 1 in. on the chart. The paper chart was 4 in. wide and 24 in. long. The graph was made by electric sparks from the metal point to the metallic drum, the

point and drum being connected to the high-tension terminals of a spark coil. This method of recording the graph permitted the lever arm to operate with a minimum of friction, and produced a chart that could be easily read or blue-printed. The chart drum was driven by weights, and timed by a clock mechanism to make one revolution per minute (Fig. 7). The vacuum line to the draft-tube terminated in a $\frac{1}{8}$-in. copper tube, inserted just below the wheel band, and pointing downward in order to avoid the centrifugal force of the water. A number of other connections were tried, but this was the only kind that would remain in place. Although it did not record the true draft head, it was the most satisfactory plan proposed for securing comparative readings.

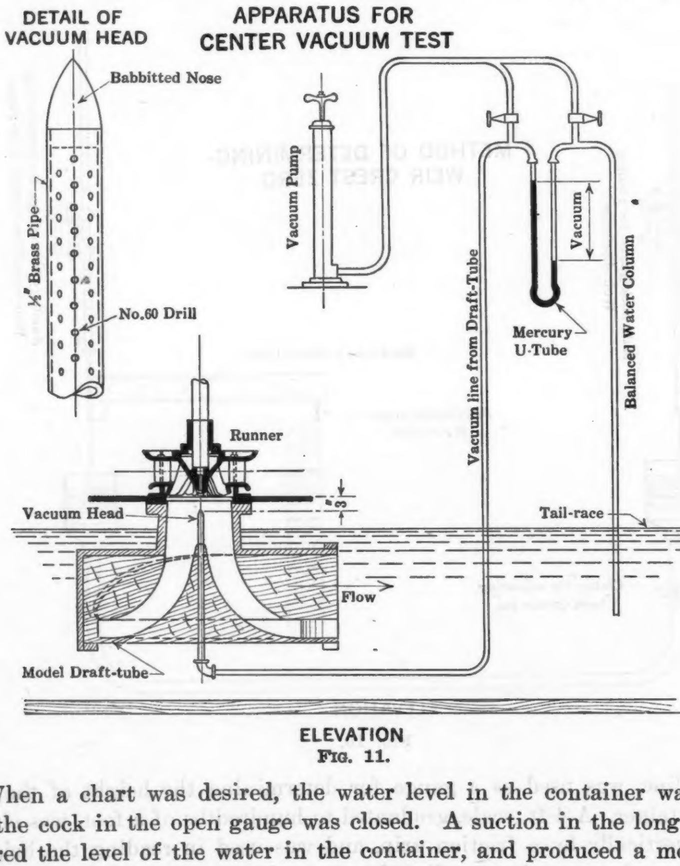


FIG. 11.

When a chart was desired, the water level in the container was adjusted and the cock in the open gauge was closed. A suction in the long glass tube lowered the level of the water in the container, and produced a movement of the float and lever arm which was recorded on the chart.

Mercury U-Tube for Determining Center Vacuum.—During the early experiments an attempt was made to determine the vacuum in the center of the draft-tubes by using an open water-column similar to that used with the automatic vacuum recorder just described. It was found, however, that the vacuum was too high to be determined by this method, and a mercury U-tube was substituted (Fig. 11). One arm of the U-tube was connected

by the vacuum pipe to a ½-in. brass tube with a perforated head and pointed nose of babbitt metal which was placed in the center of the draft-tube and 3 in. below the runner. The other arm of the U-tube was connected to a ¼-in. water line extending to a point in the discharge culvert below the surface of the tail-water. A vacuum pump, connected across the arms, was used to withdraw the air and bring the water columns on each side of the U-tube to a balance. A vacuum as high as 7½ in. was recorded with this apparatus.

Testing Methods

In making comparative tests of this nature, all conditions but one should remain the same throughout the series, if reliable data are to be obtained. In these tests, the draft-tube was the variable factor, and efforts were made to hold all other factors the same. The "human equation" was eliminated for the most part by automatic controls, and important readings were always made by the same man and sometimes checked by another.

As affecting the tube itself, a number of changes in its set-up were made, for the purpose of observing their effect on its performance (see Fig. 9 for set-up of Type B). Only a few of these results are included in this paper, as they constituted a departure from the primary purpose, which was to test the tubes as originally designed.

Number of Runs.—As noted elsewhere, five tubes were tested in the first and seven in the second series. As many as twenty-one tests were made on one draft-tube. From six to nine gate openings, depending on the shape of the performance curve, were used in each test. Five runs were made for each gate opening, to get the necessary range of speeds, so that from 30 to 45 runs were necessary to complete one test (see specimen data sheets, Tables 2 and 3). All data sheets and curves applying to a test were given a symbol denoting the number of the test, the number of the series, and the type letter of the tube. For instance, S_1-B-10 denoted the tenth test on Tube B, made in the first series.

It was found that 3 min. was the best length of time for a run. To balance the load and get a steady flow in the tail-race required 2 min., so that each complete speed run required 5 min. To change the gate opening after every fifth run required 10 min.

Check runs of each test were made. In some cases the tubes were set up a second time and check runs made to see if the manner of setting affected the results. In every case of this nature, the check results varied less than 0.2 per cent. In a number of instances check runs showed a variation of less than three-tenths of 1 rev. per min., although several changes in speed had been made in the meantime. On the whole, the entire equipment was very sensitive, and any trouble could easily be observed and remedied before the work went forward.

Routine Adjustments and Inspections.—The thrust-bearing and wheel-casing were aligned as accurately as possible by using a machinist's hand-level and plumb-bob. The 3-in. shaft was inspected frequently for alignment, and a 0.0001-in. shim feeler was tried around the babbitted steady bearing in the upper part of the wheel-casing to test for clearance. In the

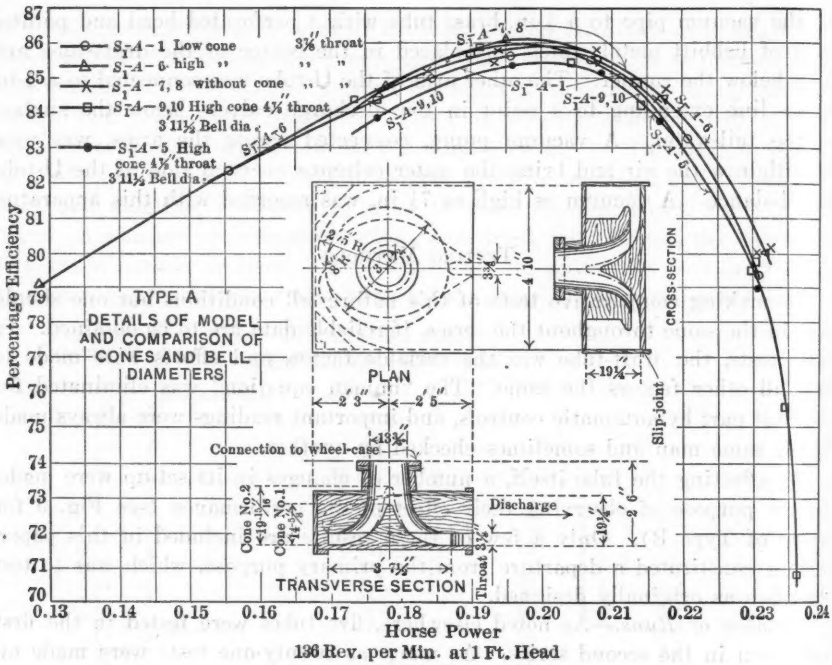


FIG. 12.

earlier tests the starting torque was determined by wrapping a cord around the small part of the shaft and noting the pull on a spring-balance scales. This method was discontinued, however, as it was found that when the shaft was in proper alignment it could easily be spun with the fingers. This fact illustrates the good adjustment of the moving parts, the combined weight of which was about 400 lb.

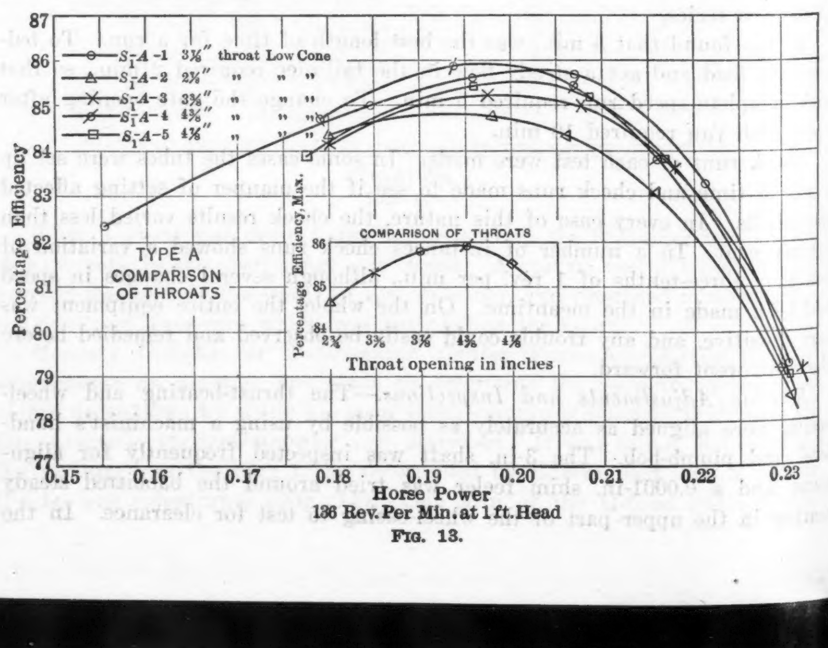


FIG. 13.

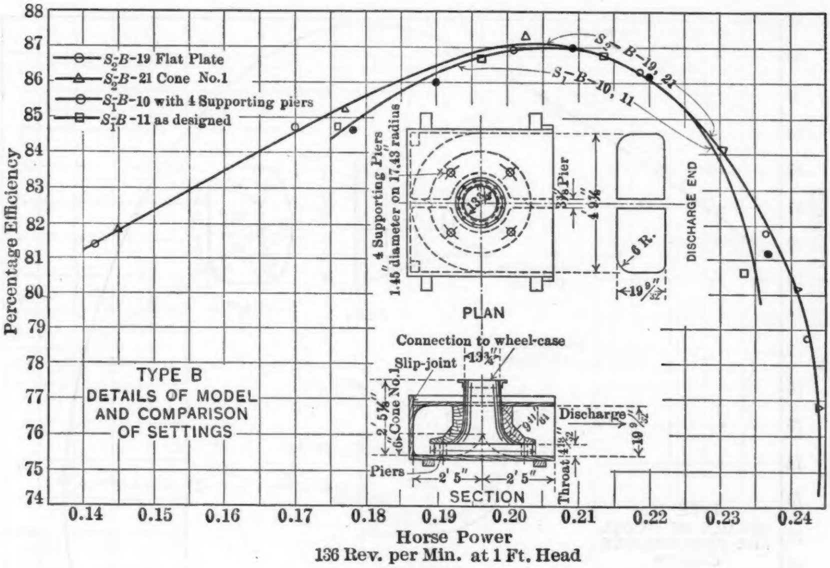

S_2-B-19 Flat Plate
S_2-B-21 Cone No.1
S_1-B-10 with 4 Supporting piers
S_1-B-11 as designed
S_2-B-19, 21
S_1-B-10, 11
Percentage Efficiency
4 Supporting Piers
1.45" diameter on 17.43" radius
3⅞" Pier
DISCHARGE END
6 R.
PLAN
Connection to wheel-case
Slip-joint
Cone No. 1
Discharge
Piers
Throat
SECTION
TYPE B
DETAILS OF MODEL
AND COMPARISON
OF SETTINGS
Horse Power
136 Rev. per Min. at 1 Ft. Head

FIG. 14.

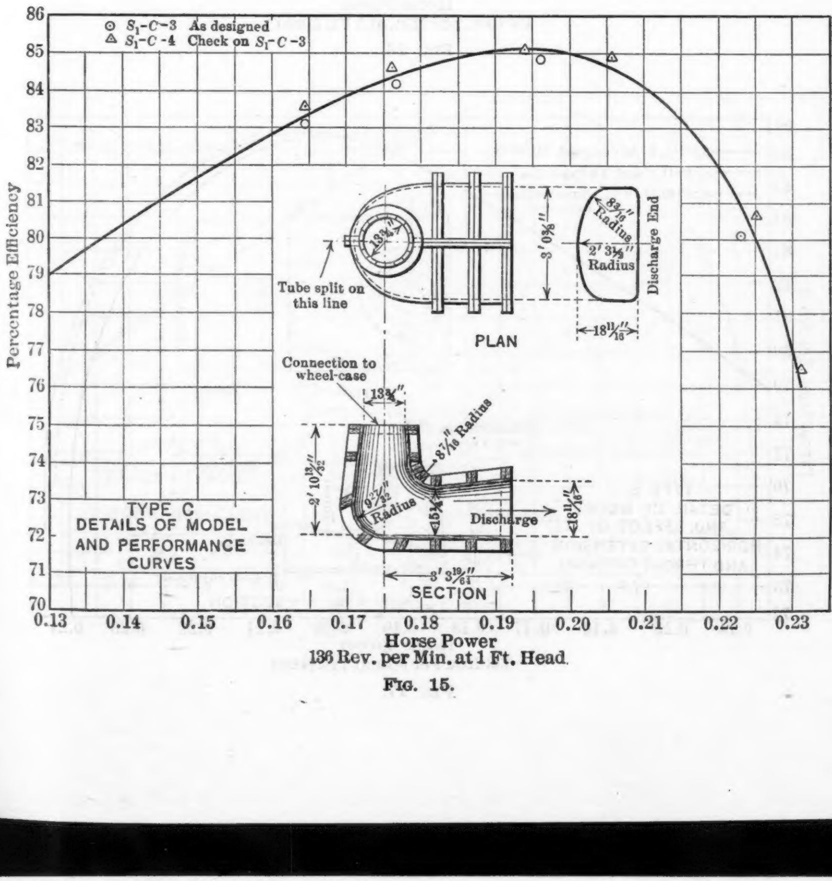

S_1-C-3 As designed
S_1-C-4 Check on S_1-C-3
Percentage Efficiency
Tube split on this line
Radius
Discharge End
PLAN
Connection to wheel-case
Radius
Discharge
SECTION
TYPE C
DETAILS OF MODEL
AND PERFORMANCE
CURVES
Horse Power
136 Rev. per Min. at 1 Ft. Head

FIG. 15.

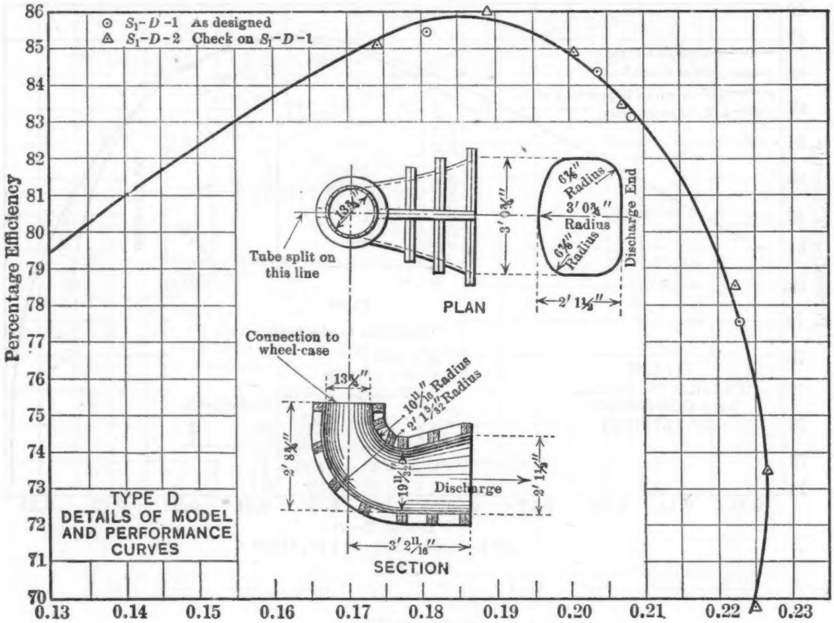

⊙ S_1-D-1 As designed
△ S_1-D-2 Check on S_1-D-1
Percentage Efficiency
Horse Power
136 Rev. per Min. at 1 Ft. Head
TYPE D
DETAILS OF MODEL
AND PERFORMANCE
CURVES
PLAN
SECTION
Tube split on this line
Connection to wheel-case
Discharge End
Discharge
Radius

FIG. 16.

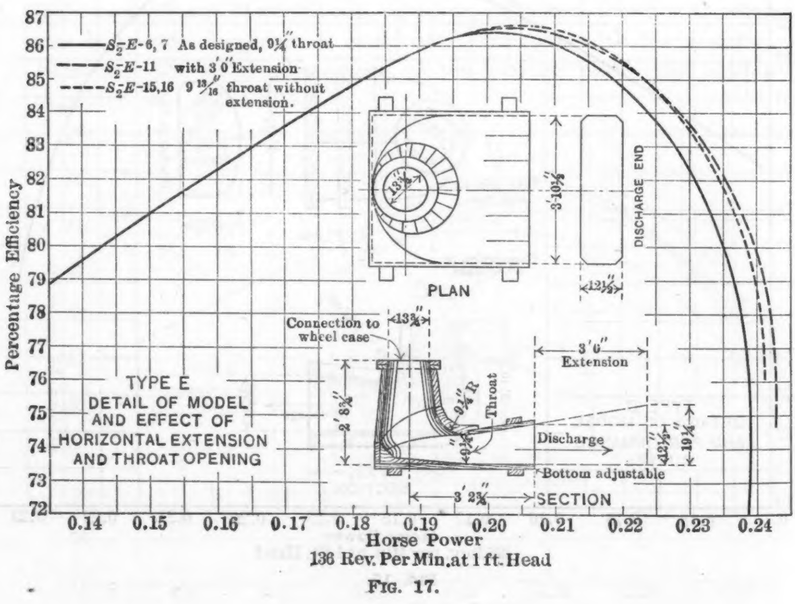

S_2-E-6, 7 As designed, 9¼″ throat
S_2-E-11 with 3′ 0″ Extension
S_2-E-15,16 9 13/16″ throat without extension.
Percentage Efficiency
Horse Power
136 Rev. Per Min. at 1 ft. Head
TYPE E
DETAIL OF MODEL
AND EFFECT OF
HORIZONTAL EXTENSION
AND THROAT OPENING
PLAN
SECTION
DISCHARGE END
Connection to wheel case
3′ 0″ Extension
Throat
Discharge
Bottom adjustable

FIG. 17.

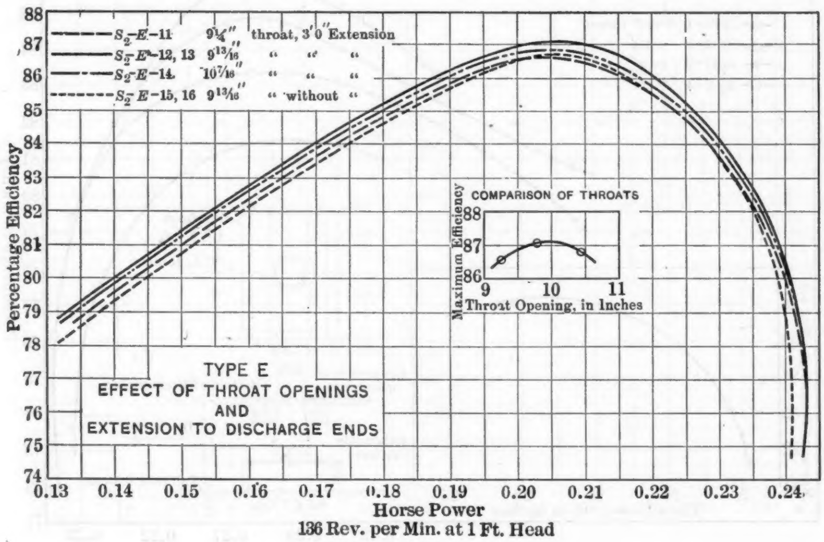

S_2-E-11 9¼" throat, 3' 0" Extension
S_2-E-12, 13 9 13/16" " " "
S_2-E-14 10 7/16" " " "
S_2-E-15, 16 9 13/16" " without "
Percentage Efficiency
COMPARISON OF THROATS
Maximum Efficiency
Throat Opening, in Inches
TYPE E
EFFECT OF THROAT OPENINGS
AND
EXTENSION TO DISCHARGE ENDS
Horse Power
136 Rev. per Min. at 1 Ft. Head

FIG. 18.

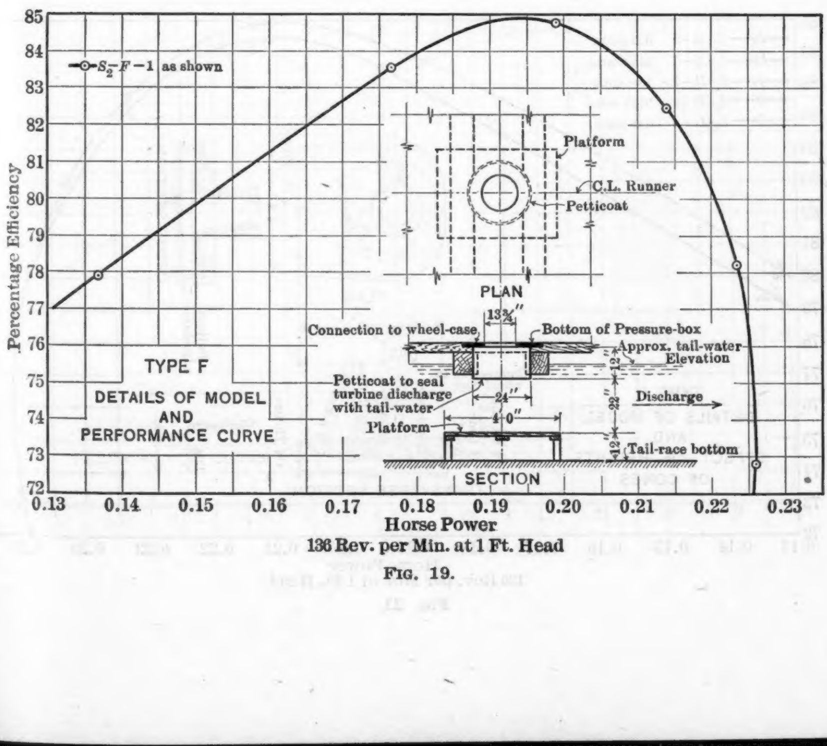
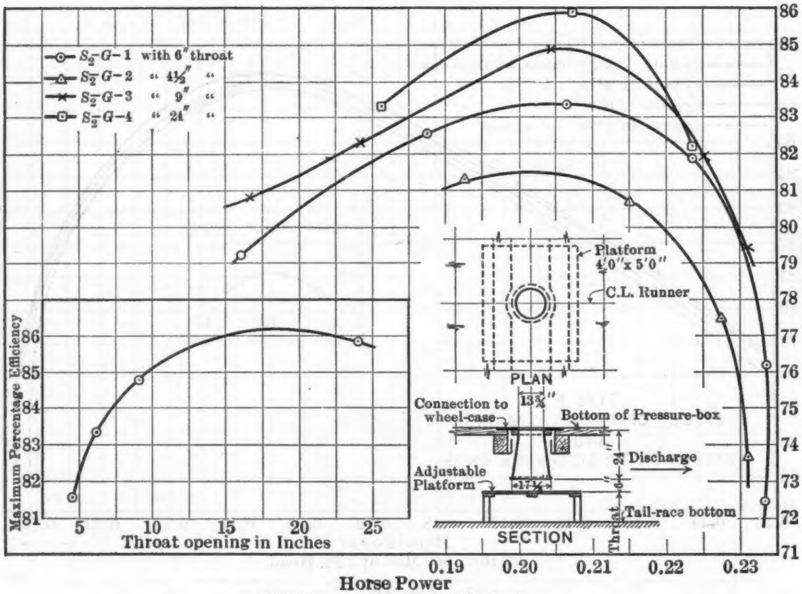
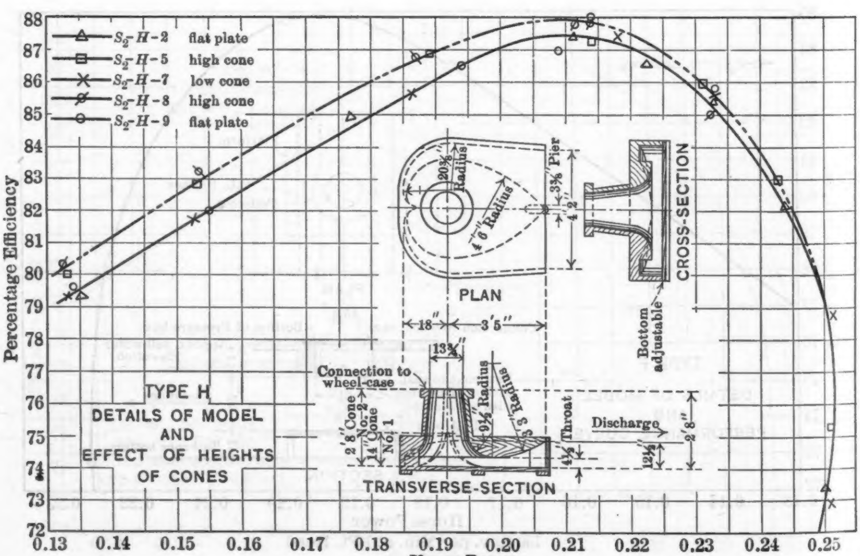

S_2-F-1 as shown
Percentage Efficiency
Platform
C.L. Runner
Petticoat
PLAN
13¾"
Connection to wheel-case
Bottom of Pressure-box
Approx. tail-water Elevation
Petticoat to seal turbine discharge with tail-water
24"
Discharge
4' 0"
Platform
Tail-race bottom
SECTION
TYPE F
DETAILS OF MODEL
AND
PERFORMANCE CURVE
Horse Power
136 Rev. per Min. at 1 Ft. Head

FIG. 19.



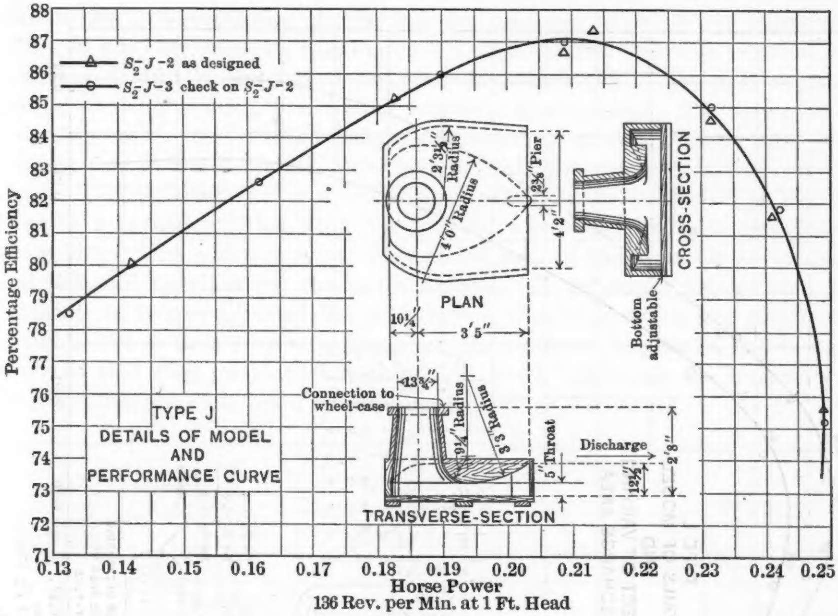
136 Rev. per Min. at 1 Ft. Head

FIG. 20.



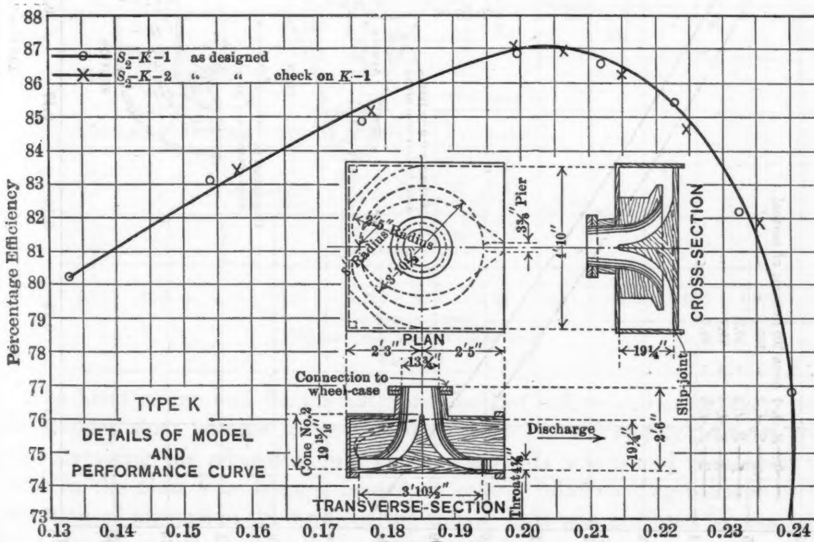
136 Rev. per Min. at 1 Ft. Head

FIG. 21.



Percentage Efficiency
S_2-J-2 as designed
S_2-J-3 check on S_2-J-2
TYPE J
DETAILS OF MODEL
AND
PERFORMANCE CURVE
PLAN
CROSS-SECTION
TRANSVERSE-SECTION
Connection to wheel-case
Discharge
Bottom adjustable
Pier
Radius
Throat
Horse Power
136 Rev. per Min. at 1 Ft. Head

FIG. 22.



Percentage Efficiency
S_2-K-1 as designed
S_2-K-2 " " check on K-1
TYPE K
DETAILS OF MODEL
AND
PERFORMANCE CURVE
PLAN
CROSS-SECTION
TRANSVERSE-SECTION
Connection to wheel-case
Discharge
Slip-joint
Cone No. 2
Pier
Radius
Throat
Horse Power
136 Rev. per Min. at 1 Ft. Head

FIG. 23.

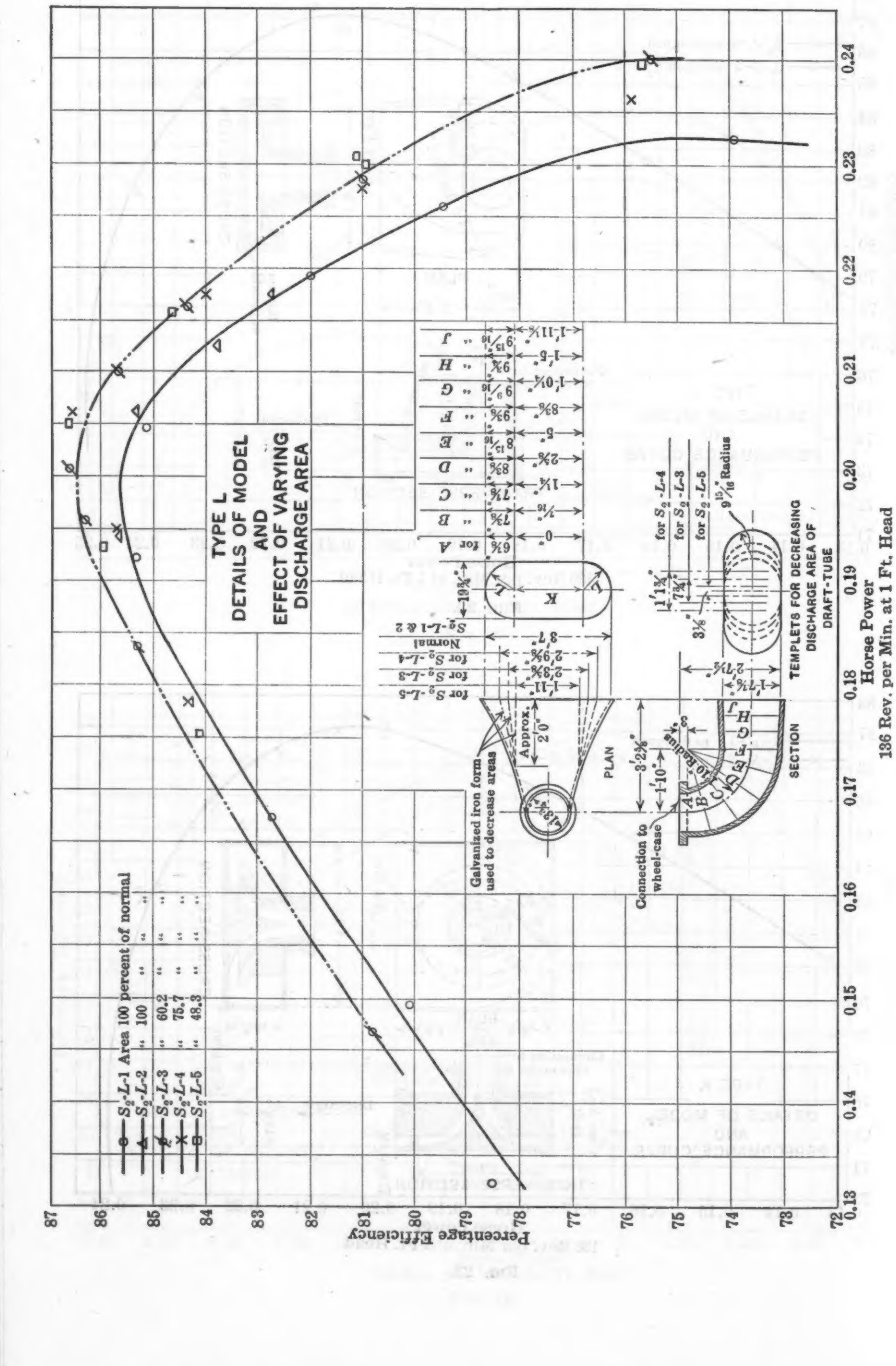
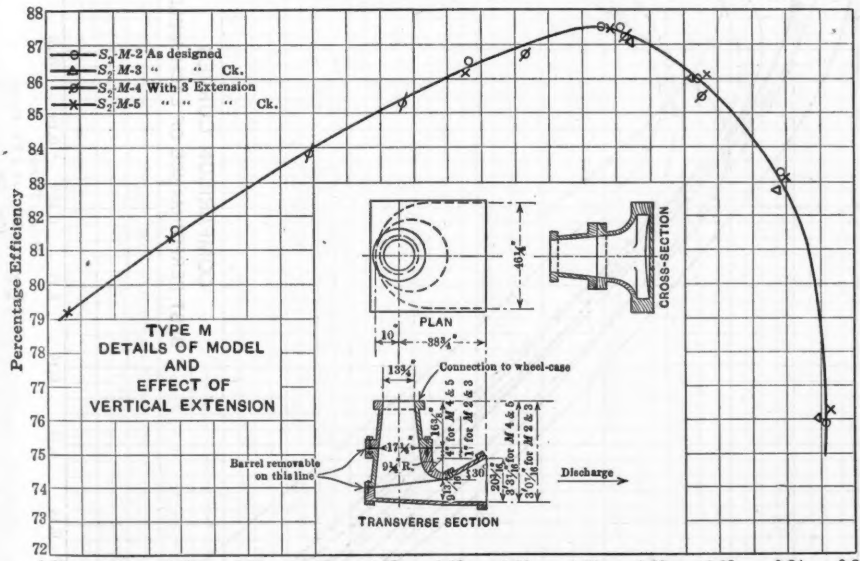

TYPE L
DETAILS OF MODEL
AND
EFFECT OF VARYING
DISCHARGE AREA
S_2-L-1 Area 100 percent of normal
S_2-L-2 " 100 " " "
S_2-L-3 " 60.2 " " "
S_2-L-4 " 75.7 " " "
S_2-L-5 " 48.3 " " "
Percentage Efficiency
Horse Power
136 Rev. per Min. at 1 Ft. Head
Galvanized iron form used to decrease areas
PLAN
SECTION
Connection to wheel-case
TEMPLETS FOR DECREASING DISCHARGE AREA OF DRAFT-TUBE
Approx. 2'0"
for S_2-L-4
for S_2-L-3
for S_2-L-5
Normal S_2-L-1 & 2

FIG. 24.

At the beginning of a test the dynamometer was leveled carefully by using turnbuckles and suspension chains, and any tendency to swing was corrected before it was lowered to its position on the shaft. After it was in position, it was moved slightly up and down with the hand. If it did not respond readily to a slight touch, the leveling operation was repeated. The zero of the weighing scales was checked before and after every test. To do this the exhaust line to the automatic valve was opened to simulate operating conditions. The dynamometer arm was disengaged from the pull-rod and swung clear in a normal position while the pull-rod was suspended by a cord from the ceiling. The scale-beam was then placed against the upper or lower stop and balanced by vibrating the scales support. If the target pointer showed a variation, it was adjusted by placing iron plate-washers on the platform. Weights of less than 1 oz. were placed on the platform frequently, and it was observed that they produced a variation in speed, indicating the importance of balancing the scale-beam exactly on the zero of the target.



Horse Power
136 Rev. per Min. at 1 Ft. Head
FIG. 25.

The head gauge and float scale were checked before and after every test. The wooden float became water-soaked to some extent, and corrections were made by using the adjusting link on the staff. It was found that when the water in the glass tube became heated, it gave a false reading because of the collection of air inside the walls of the tube. This was obviated by changing the water frequently with a thin rubber hose.

Leakage.—A constant check of the leakage from the wooden pressure-box was necessary. In the first series of tests, the leakage was determined by closing the wicket gates by placing a rubber packing gasket around them,

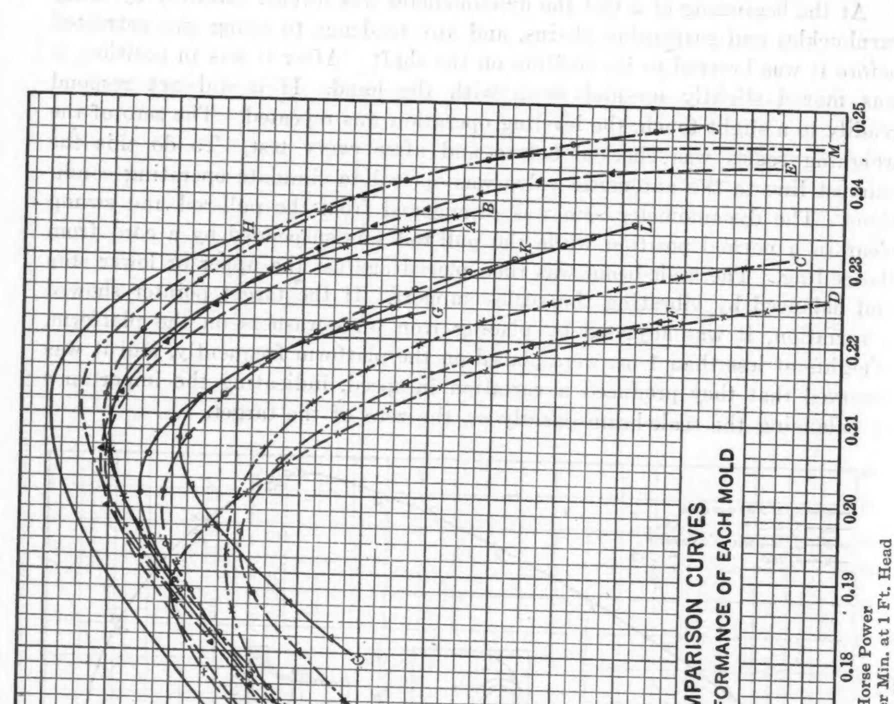

COMPARISON CURVES
BEST PERFORMANCE OF EACH MOLD
Horse Power
136 Rev. per Min. at 1 Ft. Head
Efficiency
A
B
C
D
E
F
G
H
J
K
L
M

FIG. 26.

filling the pressure-box, and then measuring the leakage flow over the weir. For this purpose an auxiliary weir was made of a 1 by 6-in. tongued-and-grooved board with a notch cut in the center to form a 12-in. contracted weir. This board was placed on top of the brass weir crest and made water-tight with sifted cinders. After the maximum leakage flow had developed, the depth on the weir was taken with a hook-gauge and the discharge computed. At intervals the leakage was caught in a tank and weighed. The weight method gave a very close check on the weir coefficient. The effective pressure head, taken simultaneously with the leakage measurements, was corrected to apply to the particular test under way. The corrections were made on the assumption that the leakage varied as $\sqrt{H}$. The variation in head between the leakage tests and the actual operating condition rarely amounted to more than 6 in. in a total of about 15 ft.

TABLE 2.—DATA OF TESTS ON ECCENTRIC TUBE, TYPE E, TEST S_2-$\grave{E}$-13, JUNE 27, 1922.

Test no.	Gate in inches.	Time of day.	Length of test, in minutes.	Counter reading.	Revolutions.	Revolutions per minute.	Brake horse-power.	Scales reading.	Load on dynamometer, h.p. 100.
				00 963					
1	1⅛	8:48:55	3	99 537	1 426	475.3	12.23	51.5	2.575
2	"	8:53:05	3	98 035	1 502	500.7	12.39	49.5	2.475
3	"	8:57:25	3	96 494	1 541	513.7	12.44	48.5	2.425
4	"	9:02:00	3	94 914	1 580	526.7	12.51	47.5	2.375
5	"	9:07:25	3	93 237	1 677	559.0	12.58	45.0	2.250
6	1¼	9:19:45	3	91 817	1 420	473.3	12.78	54.0	2.700
7	"	9:23:45	3	90 310	1 507	502.3	12.93	51.5	2.575
8	"	9:28:45	4	88 250	2 060	515.0	13.01	50.5	2.525
9	"	9:32 55	3	86 669	1 581	527.0	13.04	49.5	2.475
10	"	9:37:10	3	84 999	1 670	556.7	13.08	47.0	2.350
11	1	9:45:45	3	83 539	1 460	486.7	11.31	46.5	2.325
12	"	9:49:40	3	82 017	1 522	507.3	11.42	45.0	2.250
13	"	9:55:00	3	80 459	1 558	519.3	11.41	44.0	2.200
14	"	10:00:00	3	78 878	1 581	527.0	11.33	43.0	2.150
15	"	10:06:10	3	77 230	1 648	549.3	11.11	40.5	2.025
16	15/16	10:17:40	3	75 768	1 462	487.3	10.96	45.0	2.250
17	"	10:21:50	4	73 738	2 030	507.5	10.90	43.0	2.150
18	"	10:27:15	3	72 192	1 546	515.3	10.82	42.0	2.100
19	"	10:31:55	3	70 618	1 574	524.7	10.76	41.0	2.050
20	"	10:37:00	3	68 959	1 659	553.0	10.51	38.0	1.900
21	13/16	10:50:40	3	67 505	1 454	484.7	10.18	42.0	2.100
22	"	10:54:40	3	65 978	1 527	509.0	10.05	39.5	1.975
23	"	10:58:40	3	64 422	1 556	518.7	9.99	38.5	1.925
24	"	11:03:45	3	62 828	1 594	536.3	9.82	37.0	1.850
25	"	11:08:50	3	61 120	1 708	569.3	9.39	33.0	1.650

Before the second series was begun, the pressure-box was lined with sheet-copper and made practically water-tight, so that the leakage from the box was negligible. A test was also made in order to determine the leakage through the stuffing-box and the discharge from the automatic valve. This was so small, however, that a 6-in. contracted weir was used and the water was weighed.

Personnel for Testing.—The testing crew consisted of four men. In making a run, one man usually operated the dynamometer and a second man

changed the loads on the scales, operated the revolution counter, computed actual horse-power, and platted a field curve showing the horse-power relative to speed. The third man read the hook-gauge and set the wicket gates at the beginning of each run. The fourth man controlled the head-gauge and vacuum recorder and made computations.

TABLE 3.—RESULTS OF TESTS ON ECCENTRIC TUBE, TYPE E, TEST S_2-*E*-13, JUNE 27, 1922.

Test no.	Gate, in inches.	Time of day.	Length of test, in minutes.	Net head, in feet.	Revolutions per minute.	Brake horse-power.	Discharge, in cubic feet per second.	Percentage of efficiency.	Discharge + leakage = 0.03.	Head on weir, in feet.	Revolutions per minute.	Results at 1-ft. head, brake horse-power.	Hook-gauge, 0 = 1.002.
1	1 ⅛″	8 : 48 : 55	3	14.69	475.3	12.23	8.68	84.6	8.71	0.478	124.0	0.2172	1.475
2	"	8 : 53 : 05	3	14.68	500.7	12.39	8.73	85.2	8.76	0.474 ½	130.6	0.2203	1.476 ½
3	"	8 : 57 : 25	3	14.68	513.7	12.44	8.74	85.4	8.77	0.475	134.0	0.2210	1.477
4	"	9 : 02 : 00	3	14.67	526.7	12.51	8.76	85 8	8.79	0.475 ⅛	137.4	0.2226	1.477 ½
5	"	9 : 07 : 25	3	14.66	559.0	12.58	8.80	85.9	8.83	0.477	146.0	0.2240	1.479
6	1 ¼″	9 : 19 : 45	3	14.60	473.3	12.78	9.42	81.9	9.45	0.499 ½	123.8	0.2290	1.501 ½
7	"	9 : 23 : 45	3	14.60	502.3	12.93	9.46	82.6	9.49	0.500 ½	131.4	0.2318	1.502 ½
8	"	9 : 28 : 45	4	14.59	515.0	13.01	9.47	83.0	9.50	0.501	134.7	0.2335	1.503
9	"	9 : 32 : 55	3	14.59	527.0	13.04	9.48	83.1	9.51	0.501 ⅛	137.9	0.2340	1.503 ½
10	"	9 : 37 : 10	3	14.59	556.7	13.08	9.51	83.1	9.54	0.502 ½	145.7	0.2348	1.504 ½
11	1″	9 : 45 : 45	3	14.67	486.7	11.31	7.87	86.3	7.90	0.443 ½	127.1	0.2010	1.445 ½
12	"	9 : 49 : 40	3	14.68	507.3	11.42	7.89	86.9	7.92	0.444	132.3	0.2029	1.446
13	"	9 : 55 : 00	3	14.68	519.3	11.41	7.89	86.8	7.92	0.444	135.5	0.2028	1.446
14	"	10 : 00 : 00	3	14.68	527.0	11.33	7.86	86.6	7.89	0.443	137.5	0.2013	1.445
15	"	10 : 06 : 10	3	14.69	549.3	11.11	7.85	84.8	7.88	0.442 ½	143.3	0.1973	1.414 ½
16	15/16″	10 : 17 : 40	3	14.69	487.3	10.96	7.56	87.0	7.59	0.432	127.1	0.1945	1.434
17	"	10 : 21 : 50	4	14.69	507.5	10.90	7.55	86.6	7.58	0.431 ½	132.3	0.1936	1.433 ½
18	"	10 : 27 : 15	3	14.69	515.3	10.82	7.52	86.3	7.55	0.430 ½	134.3	0.1920	1.432 ½
19	"	10 : 31 : 55	3	14.69	524.7	10.76	7.52	85.8	7.55	0.430 ½	136.9	0.1908	1.432 ½
20	"	10 : 37 : 00	3	14.69	553.0	10.51	7.48	84.3	7.51	0.429	144.2	0.1865	1.431
21	13/16″	10 : 50 : 40	3	14.70	484.7	10.18	7.10	86.0	7.13	0.414 ½	126.3	0.1805	1.416 ½
22	"	10 : 54 : 40	3	14.70	509.0	10.05	7.06	85.3	7.09	0.413	132.6	0.1783	1.415
23	"	10 : 58 : 40	3	14.70	518.7	9.99	7.04	85.0	7.07	0.412 ½	135.2	0.1773	1.414 ½
24	"	11 : 03 : 45	3	14.71	531.3	9.82	7.01	83.9	7.04	0.411 ½	138.3	0.1740	1.413 ½
25	"	11 : 08 : 50	3	14.71	569.3	9.39	6.94	81.0	6.97	0.409	148.3	0.1664	1.411

Computations.—Actual horse-power values were reduced to a basis of 1 ft head, and a curve was also platted showing the horse-power relative to speed After each gate setting, the hook-gauge readings were tabulated and computations made for values on curves showing the efficiency relative to speed and to horse-power at 136 rev. per min. at 1 ft. head. These curves were completed before the next test began, so that a check run of any gate opening that seemed inconsistent could be made. A daily log was kept, in which were noted the type of tube under test, the changes made in setting, changes in apparatus, weather conditions, and other matters that might influence the results.

The records and computations for each test consisted of the following:

(*a*) Two field data sheets (see specimen sheets, Tables 2 and 3);

(*b*) Curve of horse-power against speed;

(*c*) Curve of efficiency against speed, at 1 ft. head;

(*d*) Curve of horse-power against speed, at 1 ft. head;

(*e*) Curve of efficiency against horse-power at 136 rev. per min., at 1 ft. head;

(*f*) Curve of hook-gauge readings;

(*g*) Data sheet showing height of tail-water above weir crest zero, height of center line of runner above tail-water, and mean vacuum, in inches of water, for each speed and gate opening;

(*h*) Automatic vacuum recorder chart.

In these curves, for purposes of comparison, all the data are platted on the basis of a uniform head of 1 ft., the usual value in such cases. The speed of 136 rev. per min. was chosen as the proper one for the particular model runner under a head of 1 ft., to give results comparable to Tube *B* (see Fig. 28), which was to be used with the full-sized runner. Figs. 27, 28, and 29 show typical curves.

Final curve sheets were prepared, using the corrected values of the various tests on each tube, and comparisons were made between various set-ups of the same tube and also between different types of similar design. Some typical results are shown in Figs. 26 and 27.

Discussion of Results

Test data, in the form of efficiency curves showing the best performance of each tube, are included as a part of this paper. There are also included curve sheets showing comparisons of tubes of different types, tubes of the same type, and different arrangements of the same tube. The curve sheet (Fig. 26), showing the best results of each of the twelve tubes under test, may be considered as a summary of the results of this investigation. An examination of these curves will show that the subject of draft-tube design is well worth serious study and investigation by engineers engaged in all branches of the water-power industry.

The relative merits of the modern tubes are not easily determined from these tests. Changes in certain features of design were made in some of the tubes, and tests were made to determine their effect on performance. As these changes were not made throughout the entire set of tubes, a flat comparison of all tubes under identical conditions is not possible, although the effect of the changes on the tube itself may be noted. The best result of the entire series was obtained from Tube *H*, provided with a high central cone extending to the top of the tube (Fig. 21). Tube *A* also gave slightly better results with a high cone (Fig. 12). The low cones, as tested with Tubes *A*, *B*, and *H*, show no advantage over the flat plate (Figs. 12, 14, and 21), except to improve the characteristics of the individual gate-opening curves. These results no doubt indicate that a high central cone is a desirable feature in a tube of this type. The effect of varying the throat discharge opening was tested with Tube *A* (Fig. 13), Tube *E* (Figs. 17 and 18), and Tube *G* (Fig. 20). The curves illustrate perhaps the best throat height for the particular tube tested, but do not afford a basis for a general conclusion regarding this feature. Tube *A* was also tested with two sizes of bell, the result indicating that the larger bell is more desirable (Fig. 12). The effect of extending down stream the horizontal discharge opening was tested with Tube *E* (Figs.

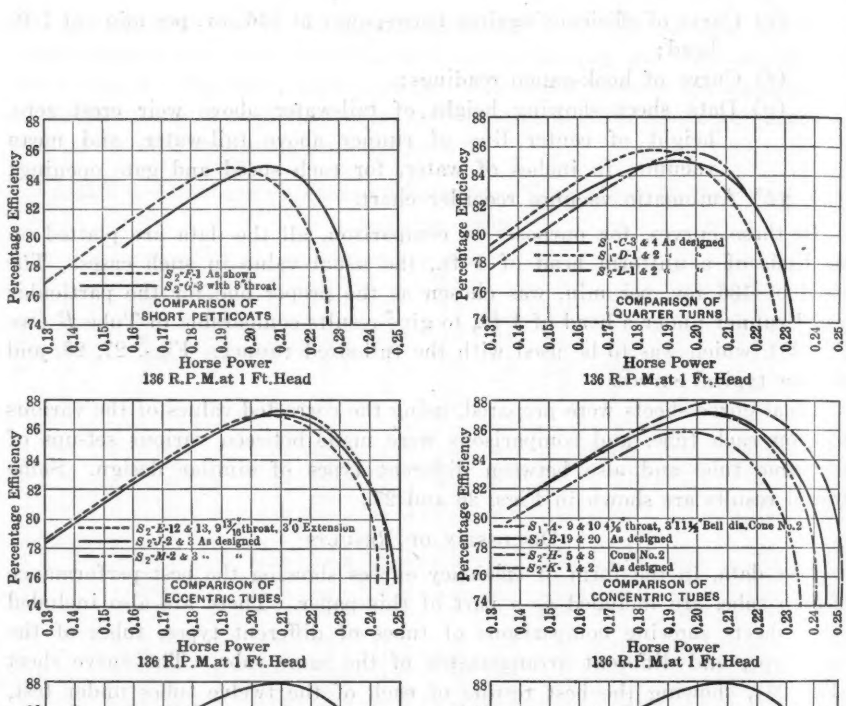

Percentage Efficiency
Horse Power
136 R.P.M.at 1 Ft.Head
S_2-F-1 As shown
S_2-G-3 with 8" throat
COMPARISON OF SHORT PETTICOATS
S_1-C-3 & 4 As designed
S_1-D-1 & 2 " "
S_2-L-1 & 2 " "
COMPARISON OF QUARTER TURNS
S_2-E-12 & 13, 9 13/16" throat, 3' 0" Extension
S_2-J-2 & 3 As designed
S_2-M-2 & 3 " "
COMPARISON OF ECCENTRIC TUBES
S_1-A- 9 & 10 4⅛" throat, 3' 11½" Bell dia. Cone No.2
S_2-B-19 & 20 As designed
S_2-H- 5 & 8 Cone No.2
S_2-K- 1 & 2 As designed
COMPARISON OF CONCENTRIC TUBES
S_2-H- 5 & 8 with Cone No.2
S_2-F- 1 As shown
COMPARISON OF TURBINE WITHOUT DRAFT TUBE AND HIGHEST CONCENTRIC TUBE
S_2-H- 5 & 8 with Cone No.2
S_2-L- 1 & 2 As designed
COMPARISON OF CONCENTRIC WITH QUARTER TURN MOLDS
LEGEND
S_1-A-9 & 10
S_1-B-16 & 17
S_1-C-3 & 4
S_1-D-1 & 2
S_1-E-1 & 2
COMPARISON OF TESTS A-B-C-D-E
S_1-A- 9 & 10 Cone No.2, 4⅛" throat, 3' 11½" Bell dia.
S_2-K- 1 & 2 " " 3' 10½" " "
with enlarged barrel
Outline S_1-A- 9 & 10
Outline S_2-K- 1 & 2
Area cut out
EFFECT OF ENLARGING VERTICAL BARREL TO CORRESPOND TO CORRECT THROAT OPENING AND BELL DIAGRAM

FIG. 27.

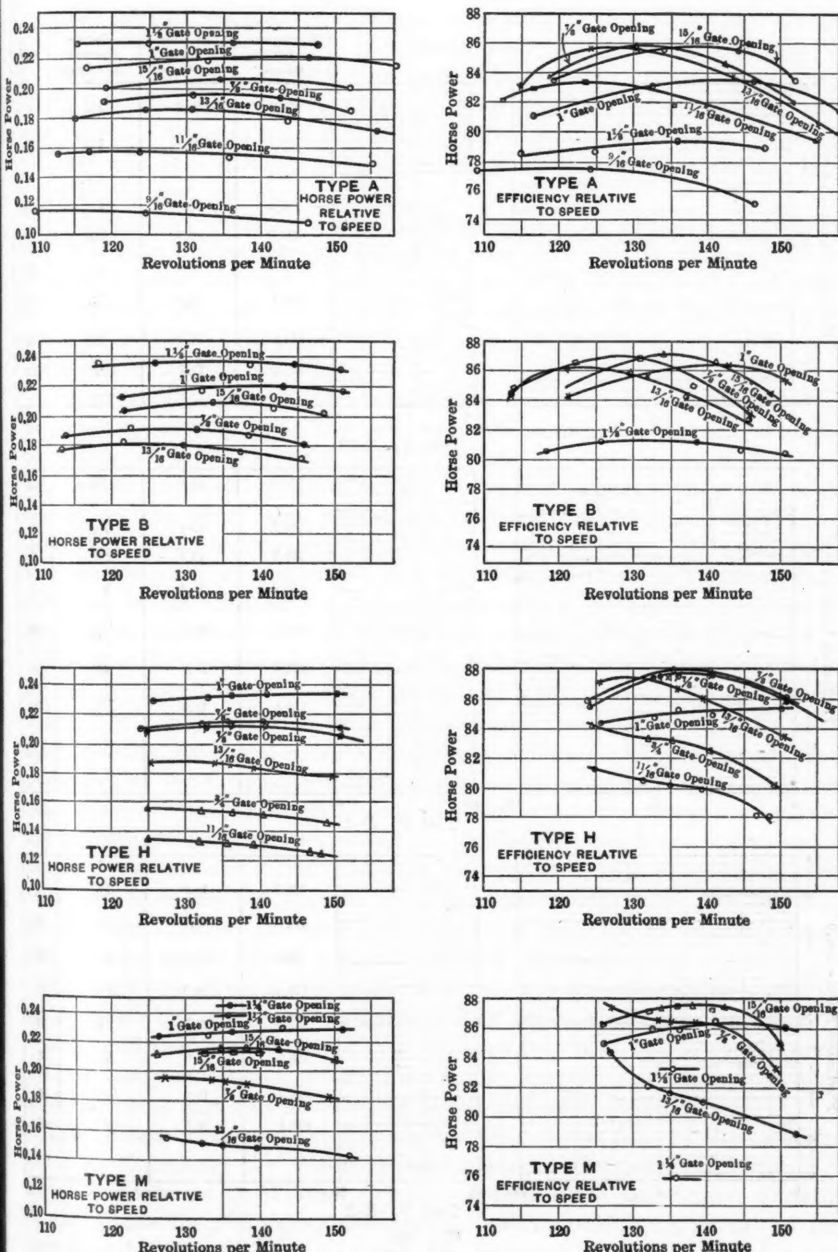
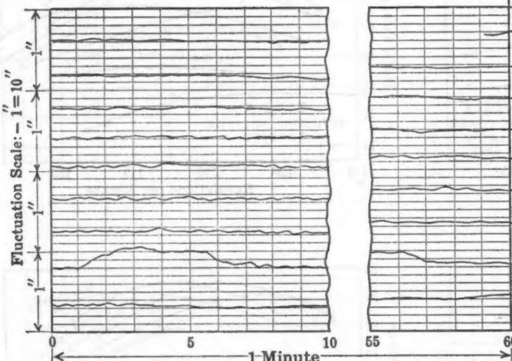
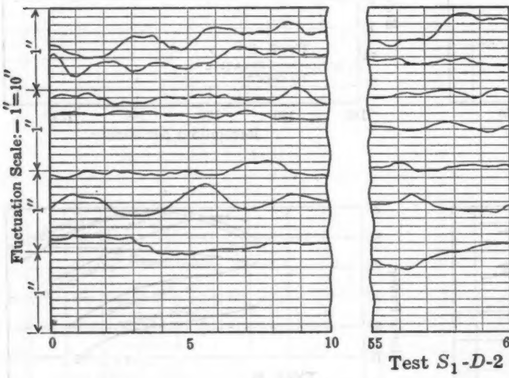

TYPE A
HORSE POWER RELATIVE TO SPEED
TYPE A
EFFICIENCY RELATIVE TO SPEED
TYPE B
HORSE POWER RELATIVE TO SPEED
TYPE B
EFFICIENCY RELATIVE TO SPEED
TYPE H
HORSE POWER RELATIVE TO SPEED
TYPE H
EFFICIENCY RELATIVE TO SPEED
TYPE M
HORSE POWER RELATIVE TO SPEED
TYPE M
EFFICIENCY RELATIVE TO SPEED
Horse Power
Revolutions per Minute

FIG. 28.



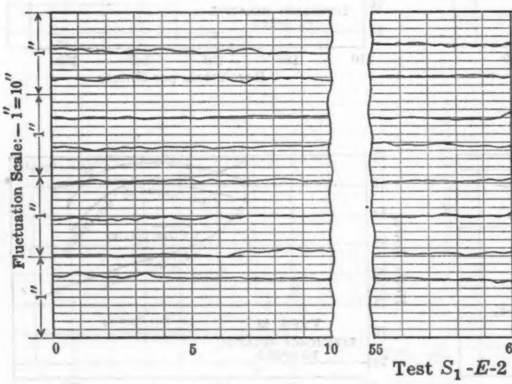
Revolutions per Minute	Vacuum in Inches of Water	Gate-Opening in Inches	Run
136.2	21.0	1⅜	6
129.7	19.3	1¼	12
135.8	18.6	1⅛	17
136.0	20.3	1	22
136.5	20.5	15/16	28
136.6	20.0	⅞	33
134.7	18.0	13/16	39
134.6	21.3	⅝	45
134.7	21.5	⅜	49

Test S_1-*A*-10



Revolutions per Minute	Vacuum in Inches of Water	Gate-Opening in Inches	Run
139.8	12.0	1⅜	33
116.2	11.4	1¼	28
138.0	12.0	1⅛	2
115.3	11.4	1	7
140.6	15.7	15/16	12
127.7	13.8	⅞	16
140.4	18.0	13/16	20

Test S_1-*D*-2



Revolutions per Minute	Vacuum in Inches of Water	Gate-Opening in Inches	Run
130.3	18.6	1⅜	3
153.0	22.8	1¼	10
139.3	21.0	1⅛	14
130.0	20.0	1	18
139.3	23.5	15/16	24
136.7	21.0	⅞	31
135.4	19.0	13/16	36
135.4	19.5	⅝	41

Test S_1-*E*-2

FIG. 29.

17 and 18), of increasing the length of the vertical barrel with Tube *M* (Fig. 25), and of varying the discharge area with Tube *L* (Fig. 24). In the case of Tube *E,* the extension is without doubt desirable, but the effect of the vertical length on Tube *M* was negligible, probably because the original design had already provided, within a practical limit of excavation, sufficient "spread" for maximum efficiency. In the case of the Tube *L,* the curve indicates that it is possible to make the taper of the discharge end too large to secure the best results.

An important feature of this investigation is the evidence obtained to the effect that fluctuations in draft-tube pressures vary in degree with the different designs, the fluctuations being least with the most efficient tubes. The vacuum charts indicate clearly the fluctuations that take place in a 1-min. period (Fig. 29). When the fluctuations were greatest, it was difficult to hold a steady load on the wheel.

The great difference in vacuum between the outer edge of the tube and the center shows that a piezometer in the customary place, that is, just below the runner on the outer wall, gives no reliable information as to the mean vacuum condition within the tube. In the case of Tube *A* (Fig. 11), a piezometer on the outer edge recorded a positive pressure of about 6 in. of water, due to the centrifugal force of the whirling water, while the center piezometer showed a vacuum of 7½ in. of mercury, or about 15 ft. of water. This high center vacuum just below the runner is no doubt the reason for the better results obtained with the high cones on Tubes *A* and *H*.

The so-called "concentric" tubes were designed with the idea of providing the freest possible flow of water concentric with the vertical axis of the runner. In the "eccentric" tubes the design is modified, with the idea of better meeting structural conditions without unduly sacrificing the hydraulic advantages of the "concentric" tube. The comparative performance of these tubes gives a basis for determining to what extent the designing engineer may be justified in using the more complex designs, which are expensive and introduce serious structural problems, as compared with the simple tubes, which are more readily adapted to design and construction. In any case, the selection of the draft-tube best suited for the type of runner adopted must still be considered as a specific problem in itself.

Although the tests discussed in this paper accomplished the particular purpose for which they were authorized, they are by no means comprehensive or final; and they are submitted with the hope that they will be supplemented by other tests made under different assumptions, from which may be deduced empirical rules that will be of value to the hydraulic engineer.

The writers wish to make acknowledgment to the Allis-Chalmers Manufacturing Company, the Wellman-Seaver-Morgan Company, and the I. P. Morris Department of William Cramp and Sons Ship and Engine Building Company, for their designs, co-operation, and valuable suggestions in connection with this work; and to thank Mr. Arthur K. Ingraham, of Worcester Polytechnic Institute, and Mr. William G. Rhinegans, of the Allis-Chalmers Manufacturing Company, for their valuable assistance in conducting these tests.

OCEAN BEACH ESPLANADE, SAN FRANCISCO, CALIFORNIA

By M. M. O'Shaughnessy,* M. Am. Soc. C. E.

Synopsis

This paper describes the Ocean Beach Esplanade at San Francisco, Calif., which was constructed in order to save the bluffs and shore road from erosion and improve the beach permanently as a pleasure resort. This was accomplished by building a stepped, reinforced concrete revetment ("bleachers") surmounted by a wall on the inshore edge and guarded at the lower edge from the undermining action of the sea by a tight cut-off wall of interlocking sheet-piles of reinforced concrete. These sheet-piles are capped by a heavy reinforced concrete beam which furnishes the support for the edge of the revetment. The inshore edge and wall are supported on, and anchored by, reinforced concrete pedestal piles, 10 ft. apart, along the structure. The revetment is divided into 20-ft. panels by heavy, sloping reinforced concrete **H**-beams, each of which is supported by the sheet-piling at one end and by a pedestal pile at the other. These beams provide for temperature movement, and furnish extra strength at the panel points. A blanket of clay, 18 in. thick, was placed beneath the revetment, in order to prevent the sand from washing out through crevices. Cross cut-offs of sheet-piling are placed about 150 ft. apart, in order to prevent progressive destruction in case one section of revetment becomes undermined. The shape of the cross-section of the revetment and the surmounting wall is designed to check the waves gradually and finally to turn them back so as to avoid splashing or inundating the walk and roadway above.

Introduction

One of the most popular sections of the system of boulevards of San Francisco is that which skirts the ocean along the westerly boundary of the city for more than 3 miles. This boulevard has a traffic width of 150 ft., and is connected on the north with Point Lobos Boulevard which passes the well-known Cliff House and Seal Rocks. (Fig. 1.) The condition of the beach prior to the construction of the Esplanade is shown by Fig. 2. Many years before the writer became City Engineer of San Francisco, the problem of protecting this boulevard, known as the Great Highway, had been attempted from time to time, but each method adopted eventually failed, due to the underwashing action of the sea. The ultimate result of this action was

Note.—Written discussion on this paper, which will not be presented at any meeting of the Society, will be closed with the **March, 1924,** *Proceedings*. When finally closed the paper, with discussion, will be published in *Transactions*.

* City Engr., San Francisco, Calif.

apparent—the Great Highway was doomed to annihilation. It became the writer's task to design and construct a barrier that would act as a permanent protection and that, architecturally, would add to, rather than detract from, the general beauty of the ocean front, and also cause no marked inconvenience to the crowds that frequent this locality.

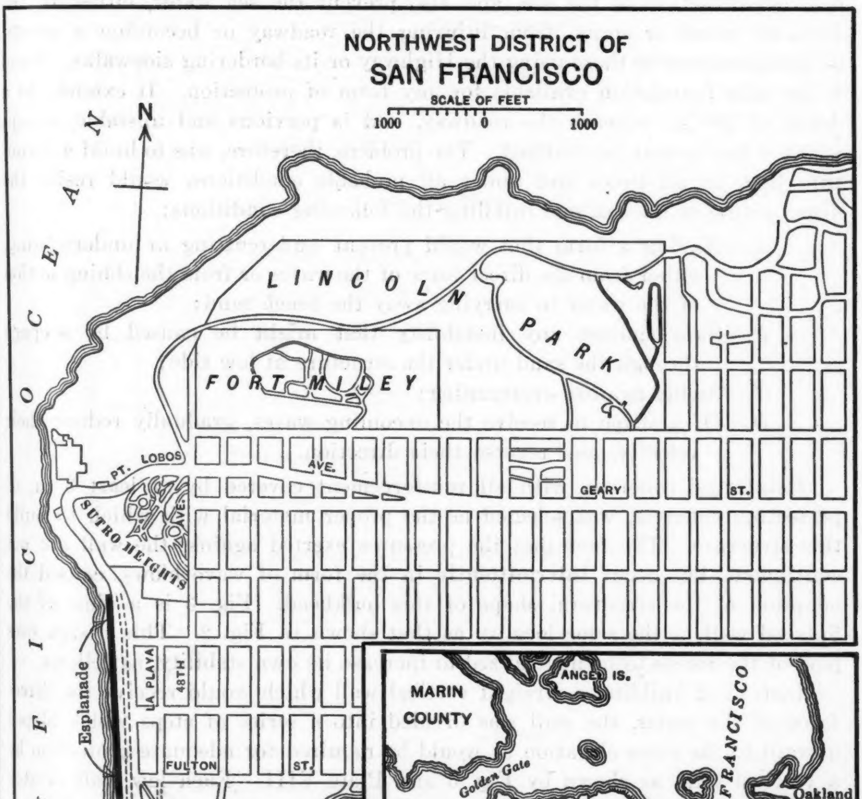


FIG. 1.

The Great Highway is 16.5 ft. above extreme high tide. The beach slopes gradually upward, terminating in a 12-ft. embankment at the westerly edge of the highway, as shown on Fig. 2. Ordinary high tide brought the sea to the

foot of this embankment, and, during extra high tides and storms, giant breakers dashed against it, at times flooding the Highway.

Design of Esplanade

The problem was to protect the highway and its embankment from the destructive action of the sea, and also prevent the sea water, either in the form of waves or spray, from injuring the roadway or becoming a source of inconvenience to those using the Highway or its bordering sidewalks. Sand is the only foundation available for any form of protection. It extends to a depth of 200 ft. beneath the roadway, and is pervious and unstable, except where it has or may be confined. The problem, therefore, was to build a structure that, at all times and under all probable conditions, would resist the direct action of the sea, one fulfilling the following conditions:

A.—Having a form that would prevent undercutting or undermining, either from the direct force of the waves or from the ebbing action of the water in carrying away the beach sand;

B.—Safe against any instability that might be caused by seepage through the sand under the structure at low tide;

C.—Stable against overturning;

D.—Of a shape to receive the oncoming waves, gradually reduce their velocity, and reverse their direction.

Reinforced concrete, with all reinforcement covered by at least 3 in. of protecting concrete, was selected as the proper material with which to build this structure. The fact that the pressures exerted against the wall are not continuous, but occur intermittently in the form of wave blows, caused the adoption of the structural shape of this bulkhead. Fig. 3 is a view of the finished wall in the same locality as that shown in Fig. 2. This design uses part of the forces to be neutralized to increase its own stability, as follows.

Instead of building a straight vertical wall which would receive the direct force of the water, the wall was divided into a series of steps which sloped upward to the same elevation as would be required for adequate protection by a vertical wall, as shown by Fig. 3 and Plate VIII. Each low wall or step receives the direct blows of the waves and causes an abrupt stopping or turning action, and, by the formation of eddies and air pockets, dissipates their force. The reaction from the force of the waves on the wall is resisted by a solidly compacted and confined foundation. Details of the front cut-off wall and the cross-wall are shown on Plate VIII. Thus, the sea itself is used to produce structural stability. The rollway shape at the highest point of the wall is for turning very high waves, and is amply reinforced to accomplish this. The horizontal force exerted against the rollway, tending to overturn the structure, is resisted both by the weight of the superimposed water and by the structure itself. The sheet-piling is integral with the structure, and its weight acts as the other side of a couple against this overturning moment.

In addition to the forces mentioned, there was also to be considered the possibility of uplift from hydrostatic pressure, due to seepage from the ocean side, and also from under the roadway on the land side. Ordinarily, the weight

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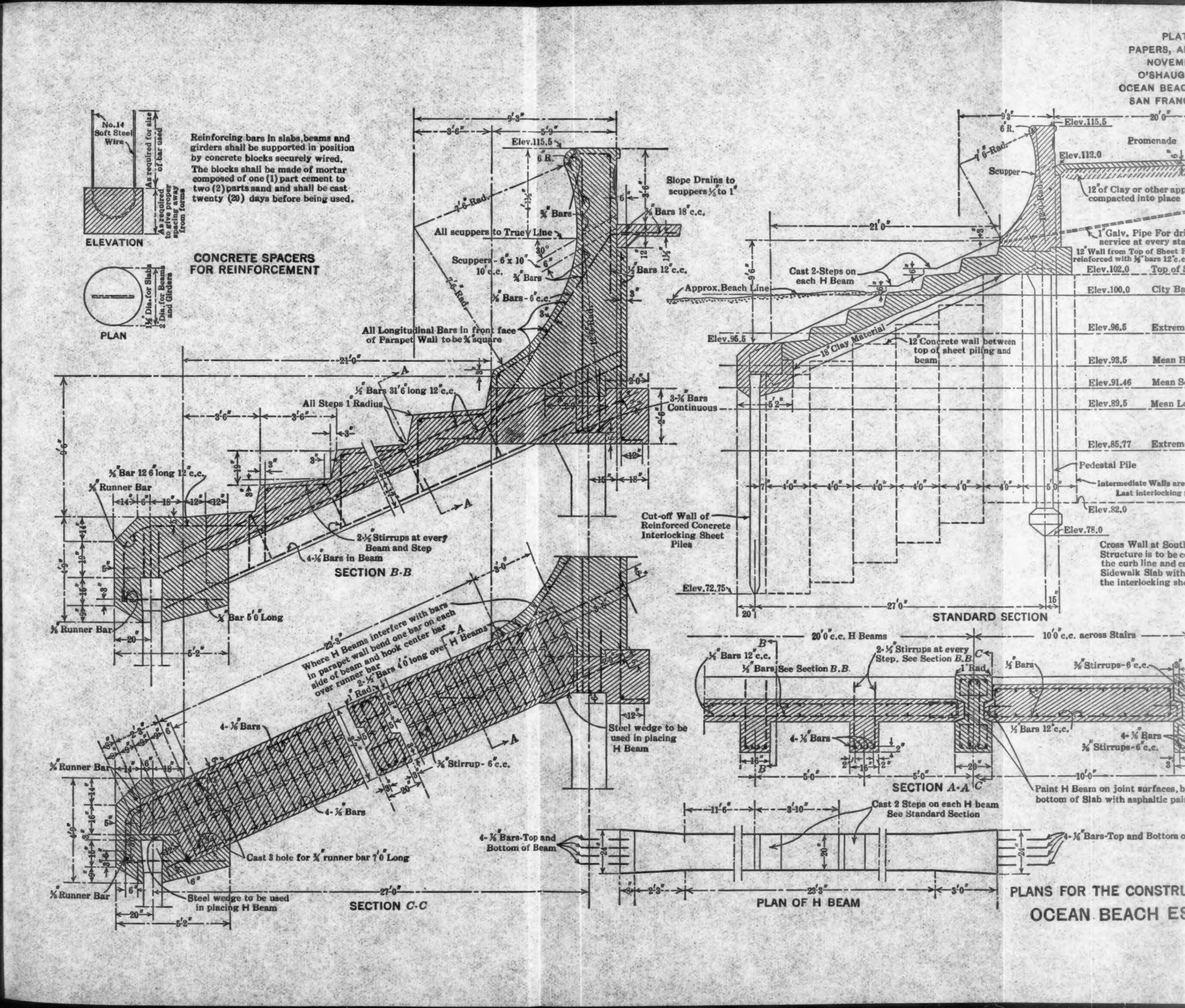

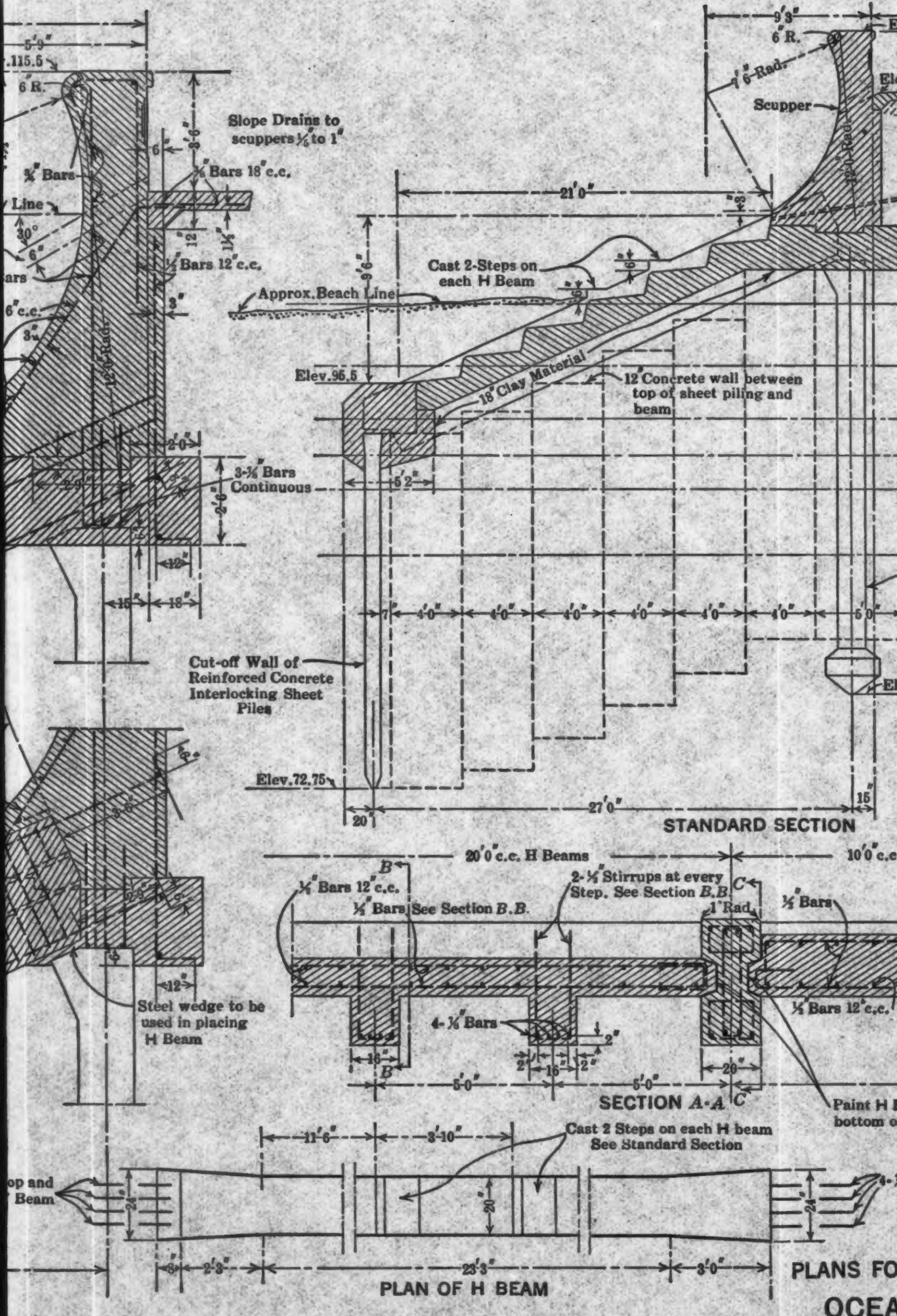

Slope Drains to scuppers ½ to 1"
⅜" Bars 18" c.c.
½" Bars 12" c.c.
Scupper
Cast 2-Steps on each H Beam
Approx. Beach Line
Elev. 95.5
18" Clay Material
12" Concrete wall between top of sheet piling and beam
3-⅞" Bars Continuous
Cut-off Wall of Reinforced Concrete Interlocking Sheet Piles
Elev. 72.75
STANDARD SECTION
20' 0" c.c. H Beams
10' 0" c.c.
½" Bars 12" c.c.
½" Bars See Section B.B.
2-½" Stirrups at every Step. See Section B.B.
1" Rad.
½" Bars
4-⅞" Bars
½" Bars 12" c.c.
Steel wedge to be used in placing H Beam
SECTION A-A
Paint H
Cast 2 Steps on each H beam See Standard Section
PLAN OF H BEAM
PLANS FO
OCEA

PLATE VIII.
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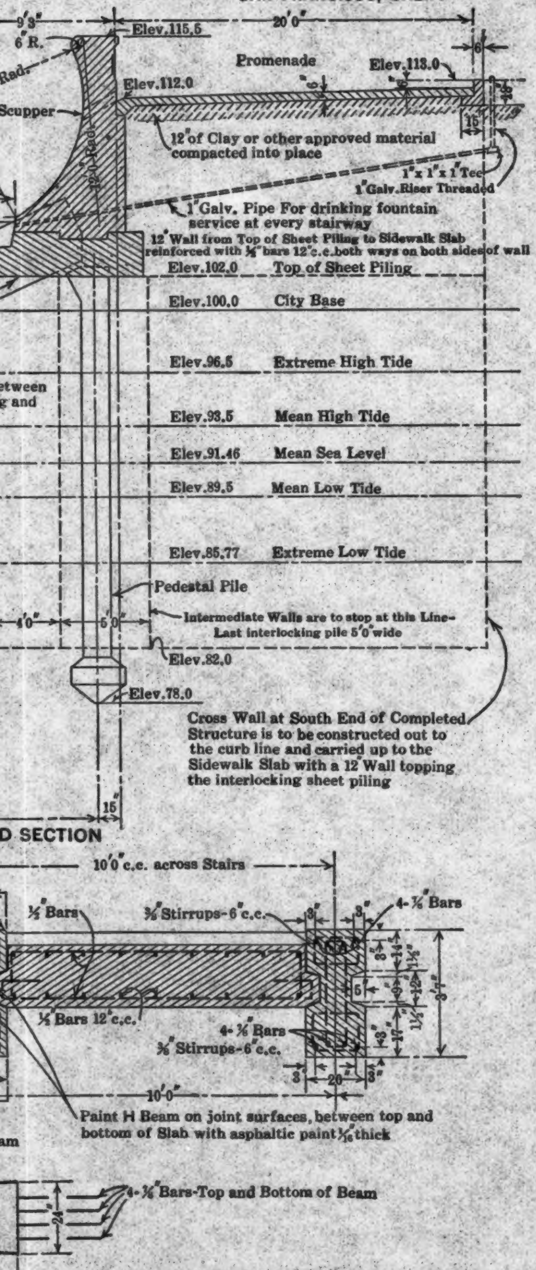
NOVEMBER, 1923.
O'SHAUGHNESSY ON
OCEAN BEACH ESPLANADE,
SAN FRANCISCO, CALIF.



PLANS FOR THE CONSTRUCTION OF THE
OCEAN BEACH ESPLANADE

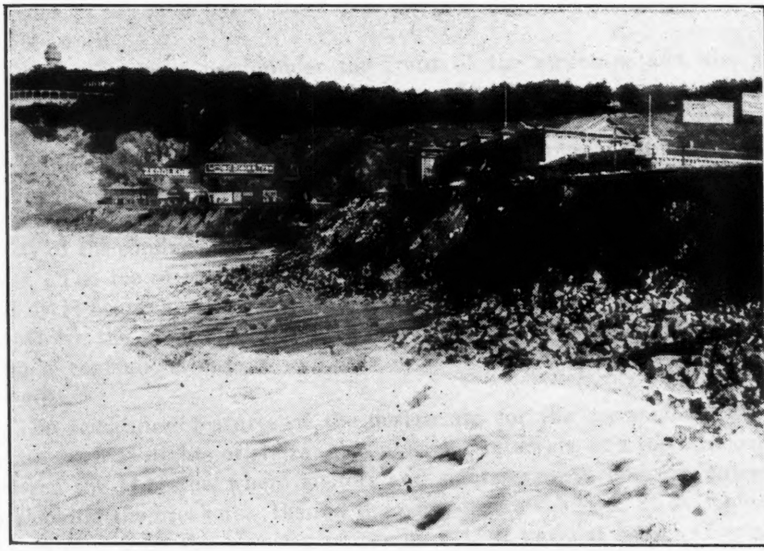


FIG. 2.—CONDITION OF BEACH PRIOR TO CONSTRUCTION OF OCEAN BEACH ESPLANADE.

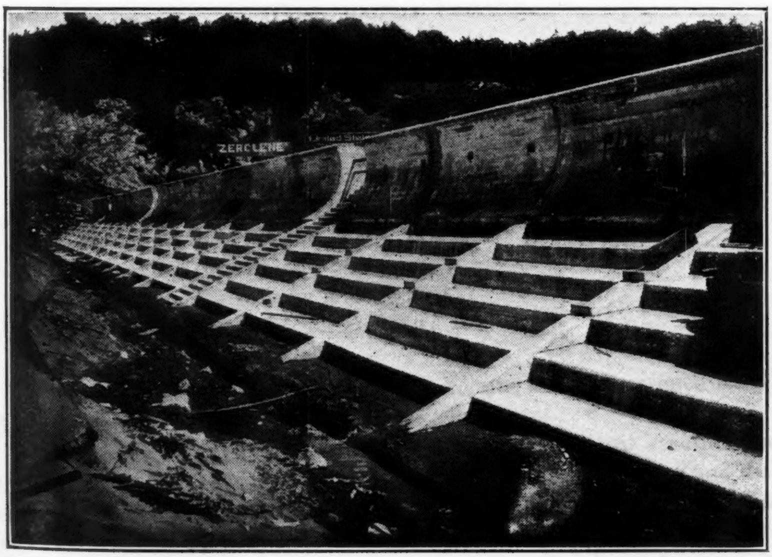


FIG. 3.—SAME LOCATION, WITH BLEACHERS AND ROLLWAY CONSTRUCTED.

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of the structure would be sufficient to overcome such uplift, but, as an added precaution, bulb or pedestal piles were used under the rear part of the structure. In order to lift such piles, the force required would be far greater than any possible uplift.

Sheet-piling was used under the front of the structure and also in the cross-walls, approximately 150 ft. apart. If a break should occur in any section, these cut-off walls are intended to confine the failure to that section. The design of the concrete pedestal piles and the concrete interlocking sheet-piles is shown on Plate IX. Clay packing of a high grade was used under the bleachers, in order to provide an impervious blanket and prevent the removal of any of the confined sand by seepage in case of a crack in the bleacher section itself. The toe of the structure is supported on interlocking sheet-piling and tied by H-beams to the monolithic cap shown on Plate VIII. It is also supported by the compacted sand on which it is constructed. The sheet-piles form a continuous cut-off wall, the base of which is 13 ft. below extreme low tide.

The remaining features of the design are for the accommodation of the public, such as flights of stairs at convenient intervals, and the bleacher effect between the H-beams, where visitors and sightseers may find convenient seats overlooking the breakers. Behind the parapet is a sidewalk, 20 ft. wide, which is separated from the highway proper by a 15-ft parking strip. Over the top of the parapet an unobstructed view of the beach beyond may be had from automobiles parked along the edge of the highway. It is intended to provide public convenience stations at intervals as the work progresses, but, at present, only one has been completed and placed in service.

Construction Features

The construction of the esplanade is a progressive procedure, and is accomplished, as follows:

1.—Casting and seasoning the pre-cast members;

2.—Placing all sheet and bulk-piles to grade and to proper line;

3.—Excavating and lagging the cap trench and operating the sump pumps;

4.—Placing the H-beams;

5.—Pouring the cap and bleachers;

6.—Pouring the rollway and parapet;

7.—Pouring the sidewalk curbing;

8.—Pouring the sidewalk.

All pre-cast members are of 1 : 1½ : 3 concrete, and are seasoned for 30 days. All monolithic concrete is a 1 : 2 : 4 mix.

On account of possible disintegration, due to chemical reactions between the ingredients of the concrete and the salts present in the water and damp air, considerable attention was given to the aggregate in order to secure a dense and well balanced concrete.

Standard brands of Portland cement were used, subject to the customary engineering tests. A satisfactory quality of hard, sharp, and clean sand was

available adjacent to the work, which contained practically no salt, and fulfilled the following screen requirements: All sand shall pass a screen of ¼-in. mesh, and not more than 50% shall pass a sieve having 50 meshes per lin. in., and not more than 10% shall pass a sieve having 100 meshes per lin. in.

The coarse aggregate selected was a river-washed gravel in the following proportions:

¼ in. to ½ in.	15%
½ in. to 1 in.	35%
1 in. to 1½ in.	50%

The water was closely regulated to keep the mix as dry as possible consistent with the satisfactory enveloping of the reinforcing bars.

The materials and workmanship are specially inspected, in order to insure the best results. All concrete is mixed, placed, and tamped carefully, so that, when the forms are removed, the structure is a finished product, true to line and grade, with no surfaces that require plastering. Special care is taken that all steel is well embedded.

This improvement is extended in units as funds become available. Thus far, 2 066 lin. ft. have been completed. The ultimate length will be nearly 3 miles, in a straight line approximately north and south, along a beautiful sand beach, the breaker line of which will be practically parallel with the structure.

The interlocking sheet-piles were cast composite with U. S. interlocking sheet-steel lagging, and were placed in sets of eleven by the aid of jet pipes to sand level. They were then driven to the required grade by jets and steam hammer, one pile always being left high to permit interlocking with the next set of eleven piles. Derrick and pile-driving rigs have been used on this work with about equal satisfaction.

Four jet pipes, 2½ in. in diameter, with ⅞-in. nozzles, were used in jetting the piles. Two pipes were placed on each side of the pile and allowed to discharge water about 3 ft. below the point of the pile. Water for jetting piles was available at a static pressure of 130 lb. per sq. in. With two jets open at the depth of the pipe in the ground, the pressure was 72 lb. per sq. in.; with four jets open at pile depth, the pressure was 54 lb. per sq. in. No special difficulty was experienced in jetting piles, except in disposing of an occasional boulder which had to be jetted to one side. Fig. 4 is a view of the jetting operation, and Fig. 5 is a view of the bleachers under construction. A finished section of the rollway may be seen at the left.

Pedestal piles were placed in the same manner as the sheet-piles. The two operations required in placing sheet-piles to grade averaged 31.9 min. per pile. The first operation was to sink the pile to sand grade with jets; the second operation was to sink the pile to the required depth with jets and steam hammer. The average quantity of water required for each sheet-pile was 3 710 cu. ft.

The average time required to place each pedestal pile to grade was 28.2 min., and the average quantity of water used was 3 190 cu. ft.

Fig. 6 shows the rollway and parapet under construction, and Fig. 7 shows the accommodation features of the Esplanade.

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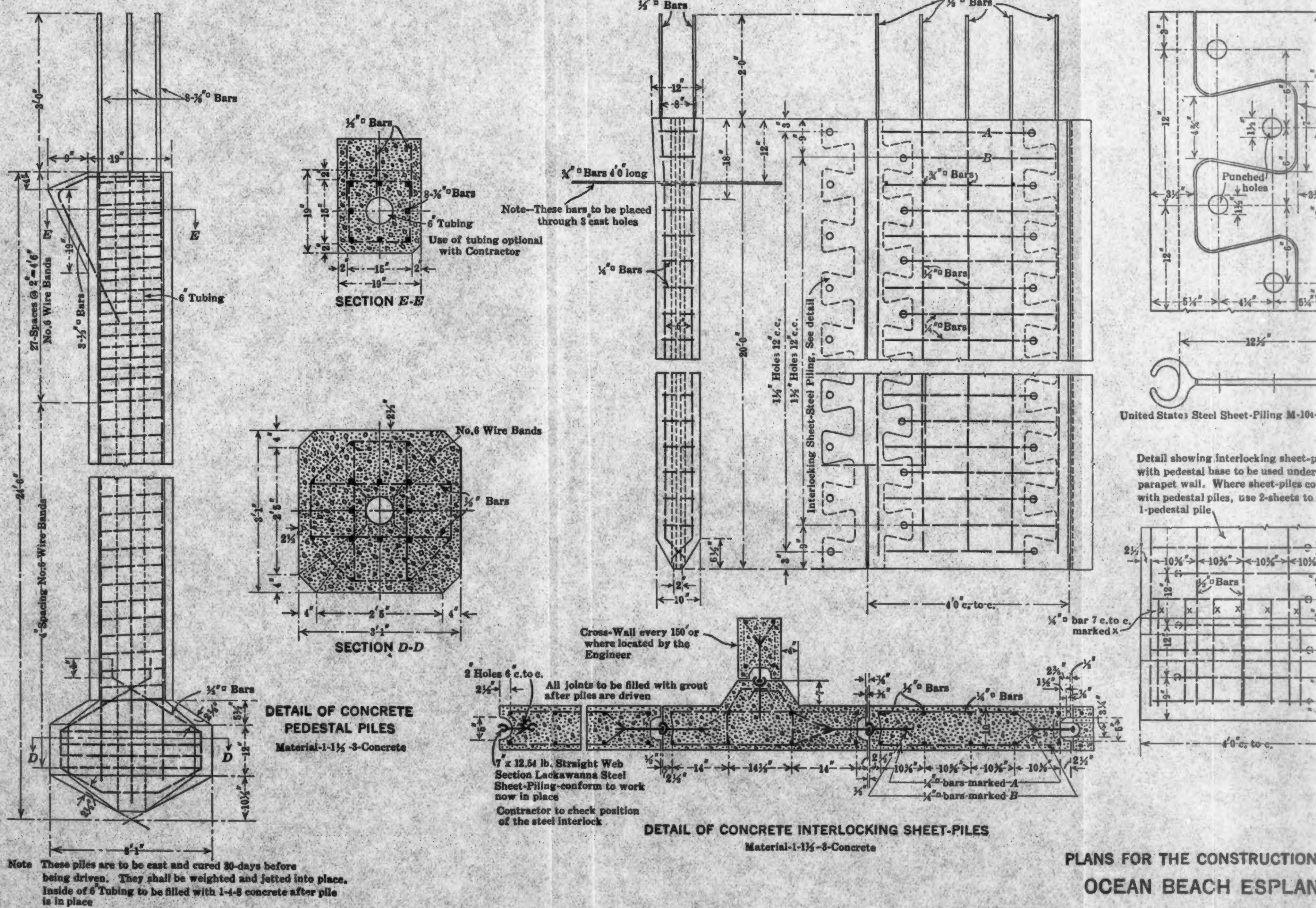

PLANS FOR THE CONSTRUCTION
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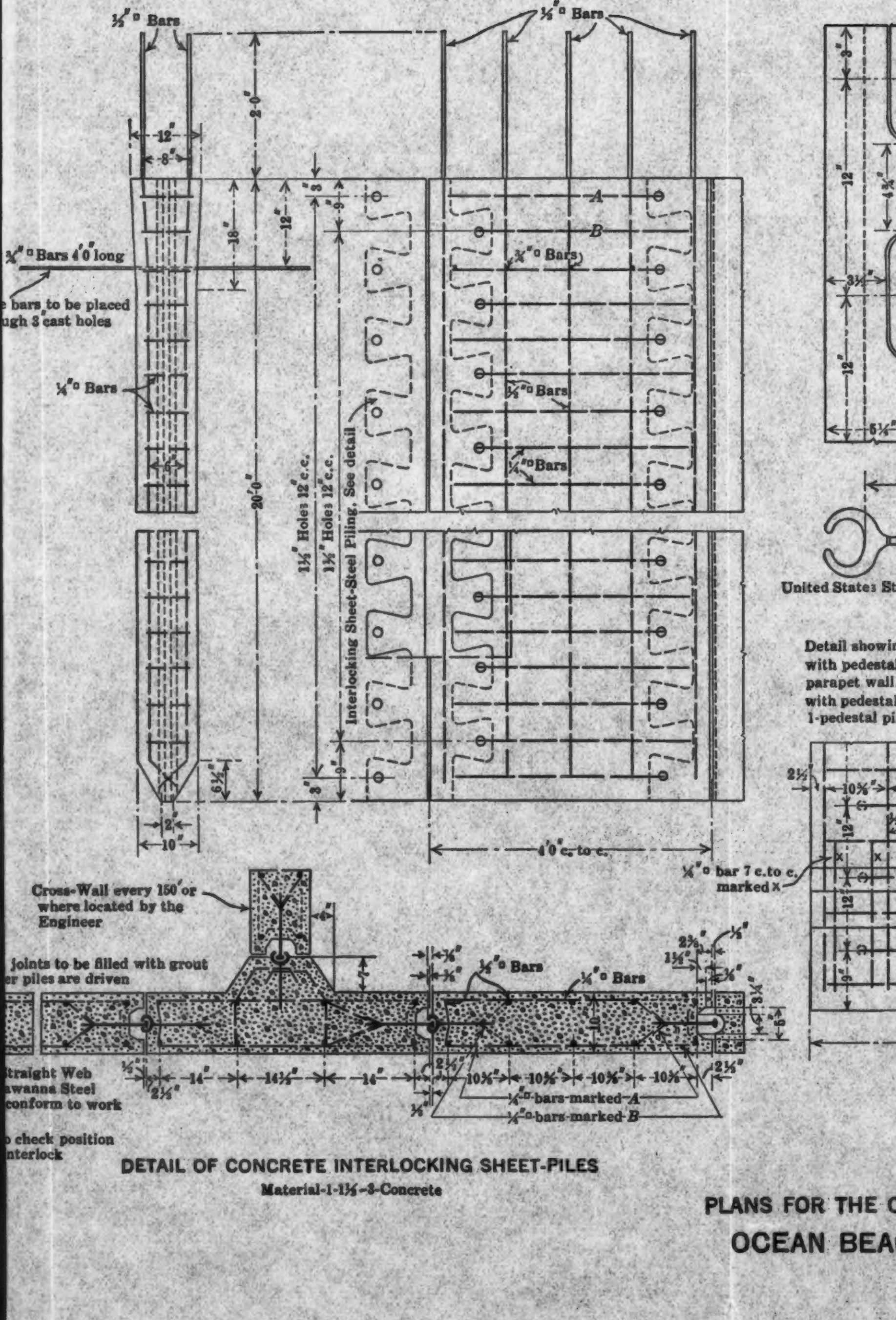

DETAIL OF CONCRETE INTERLOCKING SHEET-PILES

Material-1-1½-3-Concrete

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PLATE IX.
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O'SHAUGHNESSY ON
OCEAN BEACH ESPLANADE,
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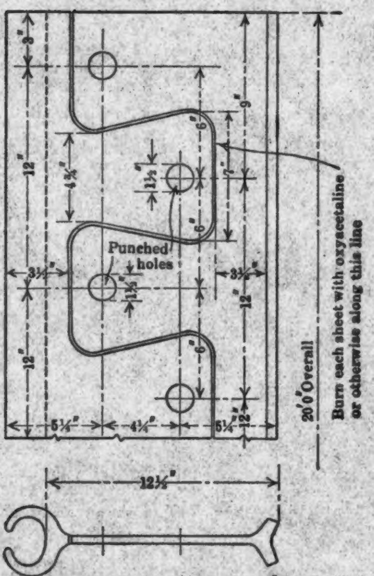
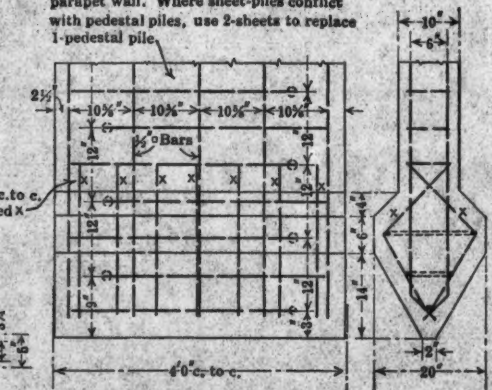


United States Steel Sheet-Piling M-104-12½" x 38-lb.

Detail showing interlocking sheet-pile with pedestal base to be used under parapet wall. Where sheet-piles conflict with pedestal piles, use 2-sheets to replace 1-pedestal pile.



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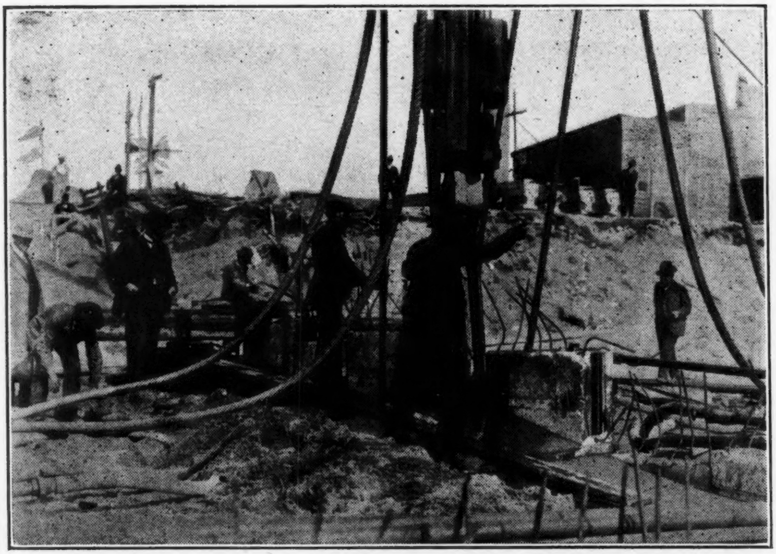

FIG. 4.—JETTING DOWN PILES, OCEAN BEACH ESPLANADE, MAY 5, 1916.

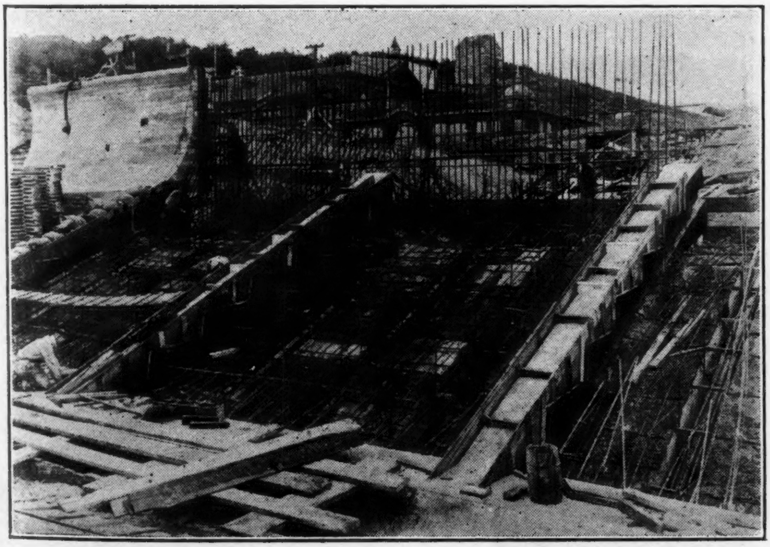

FIG. 5.—BLEACHERS, OCEAN BEACH ESPLANADE, UNDER CONSTRUCTION, OCTOBER 7, 1916.

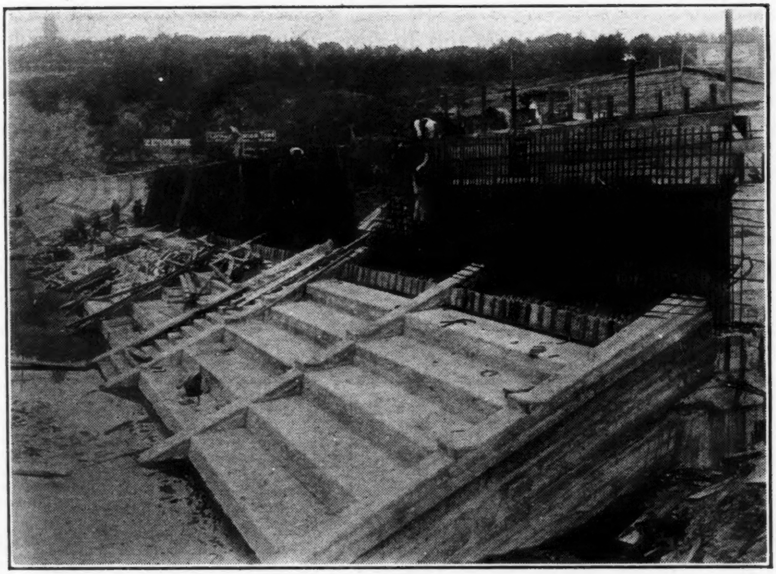

FIG. 6.—ROLLWAY AND PARAPET, OCEAN BEACH ESPLANADE, UNDER CONSTRUCTION.

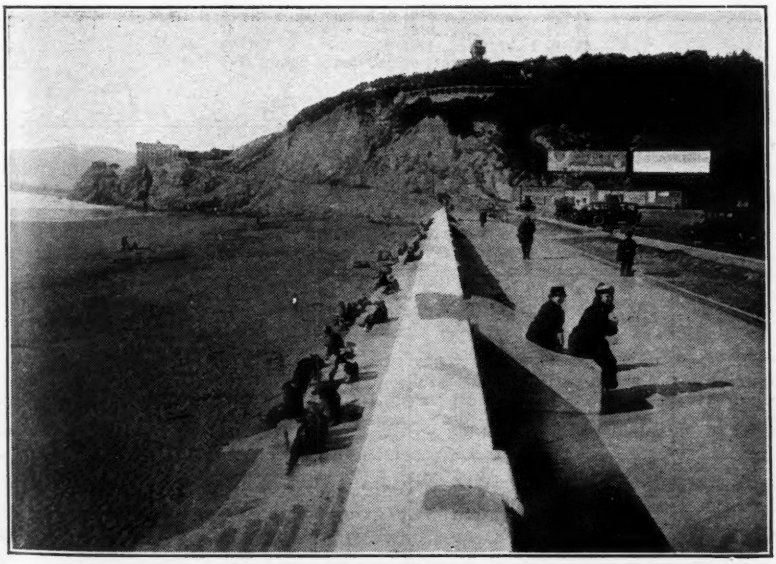

FIG. 7.—ACCOMMODATION FEATURES OF OCEAN BEACH ESPLANADE, SAN FRANCISCO, CALIF.

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Cost

Table 1 shows the construction costs of this work for three completed sections, making 1 740 lin. ft. of structures. The prices bid for the work are shown in Table 2, and the unit weights and quantities per linear foot of construction are shown in Table 3. For comparison, the costs of labor are given in Table 4 and the costs of materials in Table 5.

TABLE 1.—Cost per Linear Foot of Structure.

	Section A.	Section B.	Section C.
Plant	$ 4.56	$ 4.57	$ 7.50
General	4.18	3.62	5.18
Sheet-piles	23.35	22.04	30.21
Pedestal-piles	5.26	7.67	7.44
H-beams	7.88	8.91	8.48
Cap, bleachers	41.29	40.30	46.84
Parapet, rollway	19.29	18.84	23.70
Curb	0.80	0.72	0.80
Sidewalk	4.21	4.52	5.11
Backfill	8.98	2.75	1.22
Extra work	7.12	0.99	3.77
Overhead	5.07	7.17	4.03
Clean up	......	.....	.85
Cost to contractor	$131.94	$122.10	$145.13

Section A was constructed in 1915 .... 500 ft.
" B " " " 1916 .... 170 ft.
" C " " " 1921 .... 1 070 ft.
" D is under construction (1922) ...... 326 ft.

2 066 ft.

TABLE 2.—Prices Bid for the Work.

	Contract price per foot.	Extras.	Total.
Section A	$ 89.50	$4 865.00	$ 49 615.00
Section B	136.17	....	23 148.90
Section C	129.94	3 818.48	142 854.28
Section D	167.00	Now building	54 442.00
Total cost of 2 066 lin. ft.	......	....	$270 060.18
Average bid cost per foot for all sections	......	....	$130.72

TABLE 3.—UNIT WEIGHTS AND QUANTITIES PER LINEAR FOOT.

	Steel, in pounds.	Concrete, in cubic yards.
Interlocking sheet-steel lagging	232.5	
Sheet-piles (including cross-walls and topping)	90.7	0.74
End walls	1.2	0.018
Pedestal piles	58.0	0.242
H-beams	60.8	0.29
Parapet and rollway	116.8	1.29
Bleachers	162.1	1.94
Stairs and buttresses	11.7	0.162
Cap	29.8	0.71
Sidewalk	16.9	0.395
Totals	780.5	5.787

Total dead weight of structure per linear foot = 12 tons.

TABLE 4.—COMPARATIVE COSTS PER HOUR, OF LABOR BETWEEN SECTIONS A AND B, CONSTRUCTED IN 1915-1916, AND SECTIONS C AND D, CONSTRUCTED IN 1921-1922.

	Sections A and B, 1915-1916.	Sections C and D, 1921-1922.
Superintendence	\$1.12½	\$1.56¼
Pile driver foreman	1.00	1.15⅝
Engineer	0.75	1.00
Riggers	0.62½	1.00
Steel foreman	0.75	1.18⅞
Carpenter foreman	0.75	1.18⅞
Labor foreman	0.62½	0.87½
Finisher	0.75	1.12½
Housesmith	0.62½	0.87½
Carpenter	0.62½	1.00
Helper	0.37½	0.62½
Concrete labor	0.50	0.62½
Mixerman	0.62½	1.06¼
Common labor	0.37½	0.56¼

TABLE 5.—COMPARATIVE COSTS OF MATERIAL BETWEEN SECTIONS A AND B, CONSTRUCTED IN 1915-1916, AND SECTIONS C AND D, CONSTRUCTED IN 1921-1922.

	Sections A and B, 1915-1916.	Sections C and D, 1921-1922.
Common lumber, rough, per 1 000	\$12.00	\$18.00
Sized lumber, per 1 000	14.00	30.00
T. & G. lumber, per 1 000	18.00	32.00
Steel, average, per pound	0.03	0.0225
Cement, per sack	0.62½	0.905
Gravel, per cubic yard	2.15	3.10
Sand, per cubic yard	0.50	1.00
Auto-truck, per day	15.00	27.50
Team, 2-horse, per day	9.00	12.00
Tie wire, per coil	4.00	6.00
Fuel oil, per barrel	1.80	2.05
Gasoline, per gallon	0.12	0.23
Steel lagging, per ton	52.00	60.00
Salt water for jets, per 100 cu. ft	0.011	0.21
Compensation insurance, per \$100	7½%	8%

Conclusions

Any future extensions to the Esplanade will follow the same general design. Observations taken on Section A, built in 1915, indicate that too much importance cannot be placed on securing the proper depth to steel from the surface of the concrete, namely, at least 3 in., and on spacing the steel from the surface by means of pre-cast concrete spaces, details of which are shown on Plate VIII. In a number of places on Section A the improper placing of the steel brought it closer to the surface than the design allowed, at which points indications of rust have appeared.

All evidence points to the necessity of obtaining as impervious a mix as possible, by careful preparation and placing of materials.

THEORETICAL FREQUENCY CURVES AND THEIR APPLICATION

Discussion*

By Messrs. John Tucker, Jr., and H. Alden Foster†

John Tucker, Jr.,‡ Esq. (by letter).§—It has been stated by H. Poincare,‖ that "everybody firmly believes in it [the symmetrical (Gaussian) error function] because mathematicians imagine it is a fact of observation, and observers that it is a theorem of mathematics".

The frequency equations, derived abstractly from a purely theoretical foundation, may be applied legitimately to completely empirical formulas only under the definite condition that the frequency curve chosen represents the group of observations in the empirical data.

The general form of the frequency equation (Fig. 1 (A))¶ has been established by innumerable measurements on widely different variables. The conditions on which the general form of the frequency equation** is derived, that $\frac{d\,y}{d\,x}$ must equal zero for some finite value of x, is established empirically,†† and that $\frac{d\,y}{d\,x}$ equals zero when y equals zero, is suggested as the simplest expression of the general tendency of the empirically determined curve; but, to quote J. W. Mellor,‡‡ "in every case the number of large errors [that is, deviations from the average value] actually found is in excess of theory". The frequency curve representative of empirical data, instead of having the single peak given by the general equation of the theoretical frequency curve (Fig. 1 (A)) has the shape of a strongly dampened oscillation, with a tendency to form two smaller maxima on each side of the main peak. It follows, therefore, that the frequency curve must be applied with great caution, especially to the range beyond the portion unquestionably determined by the empirical data.

It cannot be assumed that the form of the frequency curve representing such data as stream-flow studies will be of the form indicated throughout by

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* Discussion on the paper by H. Alden Foster, Assoc. M. Am. Soc. C. E., continued from October, 1923, *Proceedings*.

† Author's closure.

‡ Pittsburgh, Pa.

§ Received by the Secretary, August 30, 1923.

‖ "Thermodynamique," Preface, Paris, 1892.

¶ *Proceedings*, Am. Soc. C. E., May, 1923, p. 829.

** *Loc. cit.*, p. 831, Equation (3).

†† In an initial consideration of the subject, and without empirical data for guidance, it would be equally probable to suppose that the value of maximum frequency or probability would be the apex of a cusp, and $\frac{d\,y}{d\,x}$ would be indeterminate.

‡‡ "Higher Mathematics for Students of Chemistry and Physics," 3d Edition, 1909, p. 514.

the end-point condition of zero probability for zero stream flow, that the tangent, $\frac{d\,y}{d\,x}$, will be zero for every point of zero probability (that is, that the y-axis is the asymptote of the frequency curve), or that y_0, a, and γ are constant for the curve which best represents a group of observations sufficiently large to determine definitely the shape of the curve.

The frequency curve selected is that which represents best the empirical data at hand, in the same manner that the best representative straight line for a group of observed values is chosen, as, for example, the equation to represent the reduction in strength of a column with increase in length. In this particular case, the straight line is selected because the tendency of the first derivative $\left(\frac{d\,y}{d\,x}\right)$ of the curve to deviate is so obscure, if existent, that it is impossible to determine.

The graphical determination of the representative frequency curve for a group of observations, unless very large, cannot be made with accuracy, because of the form of the frequency curve, with the comparatively steep peak, and particularly the two points of inflection. The duration curve, however, as plotted on probability paper,* because of its hyperbolic form, permits of making a relatively accurate graphical determination of the representative curve. By using the constants of the symmetrical form, the determination of the curve obtained from the expression, $\sqrt{\frac{\Sigma\, u^2}{n-1}}$, will give conservative results, as the curve thus obtained is a function of the squares of the variation, and hence is influenced almost entirely by the larger errors.

The most probable frequency curve which will represent a group of observed values, can be found by methods similar to those used in the application of the Gaussian normal equations to linear functions for the determination of the most probable function for which redundant observed equations of the variables have been obtained. This method, however, would be so long and tedious, with so little gain over the graphical determination of the duration curve, as to preclude its use in solving engineering problems.

Although frequency curves and their functions have had only limited engineering application, there is practically unlimited scope for their utilization in solving engineering problems of many types. Without exception, the variables used in engineering specifications, computations, or designs (physical constants, etc.), are not constant, and for the most part have considerable range of variation. The exact value of the variable can be determined by using the frequency curve, and by no other method. With the method of scientific determination of the most probable values available through the application of frequency curves, there is no reason for accepting pure guesswork—the only alternative.

Applied to the determination of positive or negative variation from the mean which will be exceeded only a predetermined small percentage of the

* *Proceedings*, Am. Soc. C. E., May, 1923, p. 846.

time, the frequency curve has endless possible application, other than its use in the study of stream flow, as given in detail by Mr. Foster. It has been used as a sound basis in determining working stresses,* and has been suggested for determining the stresses produced in structures by indeterminate impact loads, specifically to those produced in railroad track.† The influence of factors which cannot be controlled, can be determined satisfactorily, as, for example, the effect of the variation in surface finish and of this variation on the endurance strength of the object. In the manufacture of the automobile poppet-valve, the size of the finished shank can be controlled by gauges to very close dimensions, permitting a variation of less than 0.0005 in. The variation in size controls the variation in strength within approximately one-third of 1%, yet the variation in the surface finish, which cannot be controlled closely in commercial practice‡ "has an appreciable effect on the endurance strength."§

When extended, this method affords an ideal form for designating the values of the variables for acceptance qualifications in specifications, instead of fixing the upper and lower limits of the variable. The acceptance qualifications (if the symmetrical frequency form is assumed), can be placed in the form:

$$x_0 = x_m \left[1.00 \pm 0.01415\, t \sqrt{\frac{\Sigma\, u_p^{\,2}}{n-1}} \right] \| $$

The number of test specimens necessary to determine adequately the quality and uniformity of material for use in acceptance specifications can also be fixed experimentally with the aid of the frequency curve.

These few instances suggest the endless possibility in the application of this curve, and serve to extend by specific instances of widely varied application Mr. Foster's statement that "any data that can be treated as a frequency series may be studied by these methods."¶

Mr. Foster has done the Profession a service in making available in very short form the essentials of the derivation and application of the frequency curve, and especially the application of probability paper to the duration curve.

H. ALDEN FOSTER,** ASSOC. M. AM. SOC. C. E. (by letter).††—In presenting this paper to the Society, the writer was well aware that his treatment of the subject was incomplete, but he believed that the most practical way to develop the theory was to submit the paper in its incomplete form, in order that the resulting discussion might point out the proper course for further study. This decision seems to have been justified by the results.

The general theory of frequency curves is so complicated that it is essential to limit its applications to one or two types of curves. The writer limited

* *Proceedings*, Am. Soc. C. E., February, 1923, pp. 211–239.

† *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1378. Section on "Variation in Measured Stresses, and the Relation between Average and Maximum Stress" in the Second Progress Report of the Special Committee to Report on Stresses in Railroad Track.

‡ *Technology Quarterly* (Boston), March, 1899. Sondericker found that a cut (or rather scratch) of 0.003 in. reduced the endurance strength 40 per cent.

§ *Bulletin No. 124*, Eng. Series, University of Illinois.

‖ *Proceedings*, Am. Soc. C. E., February, 1923, p. 213.

¶ *Proceedings*, Am. Soc. C. E., May, 1923, p. 854.

** Structural Designer, Dwight P. Robinson & Co., New York, N. Y.

†† Received by the Secretary, September 8, 1923.

his study to a curve of Type III, as that best adapted to engineering work. A curve of Type III also offered a special advantage in that it involved the use of the second and third moment coefficients only, which greatly simplified the solution of the equation.

Mr. Hall* has suggested a modification of Type I curve to eliminate the fourth moment coefficient. This seems to the writer to be entirely suitable for practical use. Before discussing the use of this curve, it will be well to analyze its equation briefly.

Analysis of Type I Frequency Curve.—The general equation for this curve, as given by Mr. Hall, is:

$$y = y_0 \left(1 + \frac{x}{a_1}\right)^{m_1} \left(1 - \frac{x}{a_2}\right)^{m_2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

When $c.\,s. = 0$, $\beta_1 = 0$; and $m_1 = m_2 = \frac{1}{2}(r - 2)$; also, $a_1 = a_2$. Therefore, Equation (10) becomes,

$$y = y_0 \left(1 - \frac{x^2}{a_1^2}\right)^{m_1}$$

which is the equation for Type II frequency curve; that is, Type II is a special case of Type I, for a coefficient of skew of zero.

Equation (10) may be simplified by substituting:

$$z = \frac{a_1 + x}{a_1 + a_2} = \frac{a_1 + x}{b}$$

Then,

$$a_1 + x = b\,z\,;\; a_2 - x = b\,(1 - z)$$

and,

$$y = y_0 \left(\frac{b\,z}{a_1}\right)^{m_1} \left(\frac{b\,(1 - z)}{a_2}\right)^{m_2} = \frac{y_0\, b^{m_1 + m_2}}{a_1^{m_1}\, a_2^{m_2}}\, z^{m_1}\,(1 - z)^{m_2}.$$

Substituting the value of y_0 given by Mr. Hall, the Type I curve may be reduced to the form:

$$y = \frac{N}{b}\,\frac{\Gamma(r)}{\Gamma(m_1 + 1)\,\Gamma(m_2 + 1)}\, z^{m_1}\,(1 - z)^{m_2} \quad \ldots\ldots\ldots (11)$$

The corresponding duration curve will equal $\int_{-a}^{x} y\,d\,x$. As $d\,x = b\,d\,z$, the equation of the duration curve becomes:

$$y = \frac{N\,\Gamma(r)}{\Gamma(m_1 + 1)\,\Gamma(m_2 + 1)} \int_0^z z^{m_1}\,(1 - z)^{m_2}\,d\,z \quad \ldots\ldots\ldots (12)$$

Equations (11) and (12) cannot be solved until the constants b, m_1, and m_2 are determined; and in the general form of Type I curve (Equation (10)), these constants must be calculated from the data by means of the coefficients, β_1 and β_2.

By using the modified Type I frequency curve, suggested by Mr. Hall, the coefficient, β_2, is eliminated, and, as Mr. Hall has shown, all the constants may be computed from the coefficient of variation, *c. v.*, and the coefficient of skew, *c. s.* Considering the formulas for these constants as given by Mr. Hall, it will be well to examine into the influence of the *c. v.* on the constants.

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1751.

Taking *c. s.* as a constant, it will be seen that a_1, a_2, and d, vary directly with *c. v.*, m_1 and m_2 are constant, and y_0 varies inversely with *c. v.*

As,

$$z = \frac{a_1 + x}{a_1 + a_2} = \frac{(a_1 + d) + (x - d)}{a_1 + a_2}$$

z is a constant when $x - d$ varies directly with *c. v.* Then, according to Equation (12), the ordinate of the duration curve will remain constant.

The origin of this frequency curve is at the mode, or a distance of d below the mean; therefore, the quantity, $x - d$, represents the value of the variation from the mean. The previous considerations result in this important fact: In the modified Type I duration curve, for a given coefficient of skew, and a given percentage-of-time, the variation from the mean will be proportional to the coefficient of variation. This is a characteristic already noted in the case of the Type III duration curve.*

Solution of Modified Type I Duration Curve.—Equation (12) can only be integrated directly when m_1 and m_2 are integral numbers. This results in three possible cases:

Case 1.—When $c.\,s. = 0$;

$$m_1 = m_2 = 2.$$
$$a_1 = a_2 = c.\,v.\ \sqrt{7}.$$
$$d = c.\,v. \times c.\,s. = 0.$$

$$x - d = b\,z - (a_1 + d) = c.\,v.\ (2\sqrt{7}\,z - \sqrt{7}) = c.\,v.\ \sqrt{7}\,(2\,z - 1).$$

By Equation (12),

$$y = \frac{N\,\Gamma\,(6)}{\Gamma\,(3)\,\Gamma\,(3)} \int_0^z z^2\,(1 - z)^2\,d\,z$$

which reduces to

$$y = N\,z^3\,(10 - 15\,z + 6\,z^2) \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

Case 2.—When $c.\,s. = \dfrac{1}{2}\sqrt{\dfrac{7}{8}} = 0.4677$:

$$m_1 = 1. \quad m_2 = 4 - m_1 = 3.$$

$$a_1 = c.\,v.\,.\,\frac{3}{2}\sqrt{\frac{7}{8}}.$$

$$a_2 = c.\,v.\,.\,\frac{9}{2}\sqrt{\frac{7}{8}}.$$

$$x - d = c.\,v.\left(6\sqrt{\frac{7}{8}}\,z - 2\sqrt{\frac{7}{8}}\right) = c.\,v.\ 2\sqrt{\frac{7}{8}}\ (3\,z - 1).$$

By Equation (12),

$$y = \frac{N\,\Gamma\,(6)}{\Gamma\,(2)\,\Gamma\,(4)} \int_0^z z\,(1 - z)^3\,d\,z$$

which reduces to

$$y = N\,z^2\,(10 - 20\,z + 15\,z^2 - 4\,z^3) \ldots\ldots\ldots\ldots (14)$$

* *Proceedings*, Am. Soc. C. E., May, 1923, p. 840.

Case 3.—When $c.\,s. = \sqrt{\frac{7}{5}} = 1.183$:

$$m_1 = 0. \quad m_2 = 4.$$

$$a_1 = 0. \quad a_2 = c.\,v.\ 6\sqrt{\frac{7}{5}}.$$

$$x - d = c.\,v.\sqrt{\frac{7}{5}}\ (6\,z - 1).$$

By Equation (12),

$$y = \frac{N\,\Gamma\,(6)}{\Gamma\,(1)\,\Gamma\,(5)}\int_0^z (1 - z)^4\,dz.$$

which reduces to

$$y = N\,z\,(5 - 10\,z + 10\,z^2 - 5\,z^3 + z^4) \ldots\ldots\ldots\ldots (15)$$

As,

$$m_1 = 2 - \frac{6\,(c.\,s.)}{\sqrt{4\,(c.\,s.)^2 + 7}},\ c.\,s. = \frac{(2 - m_1)\sqrt{7}}{2\sqrt{5 + 4\,m_1 - {m_1}^2}}$$

Therefore, when $m_1 = -1$, $c.\,s.$ = infinity. When m_1 is greater than $+2$, $c.\,s.$ is negative.

It is evident, therefore, that the only integral values of m_1 that can be used, are covered by the three cases already considered. For any value of m_1 that is not integral, Equation (12) must be solved by some method of approximate integration, such as that mentioned by the writer.*

The writer has used Equations (13), (14), and (15) to obtain a table of factors for the modified Type I duration curve. The curve for $c.\,s. = \frac{5}{2}\sqrt{\frac{7}{11}} = 1.9943$, ($m_1 = -\frac{1}{2}$) was also solved by approximate integration. From the resulting curves, intermediate values for the various percentages-of-time and coefficients of skew were obtained by a process of interpolation. The factors are given in Table 11, which corresponds to Table 2,† except that the results are carried out to extreme values of the percentages-of-time.

In Fig. 16 are plotted the duration curves for Type I (modified) and Type III, for $c.\,s. = 0$, and $c.\,s. = 2$. It will be seen that the two types do not differ greatly between percentages-of-time of 5 and 95. The Type III curve, however, will give larger values at the upper end and smaller values at the lower end. This is what should be expected, because of the fact that the Type I curve has a definite upper limit as well as a lower limit, while the Type III curve is limited only at the lower end.

In order to extend the field of usefulness of Table 2, the writer has also computed an extension of the curves for Type III, with the results given in Table 12. It will be seen that Tables 2, 11, and 12, furnish a means of plotting duration curves corresponding to the Types I, II, III, and VII frequency curves. They, therefore, cover all cases that are likely to be needed in practical engineering work.

* *Proceedings*, Am. Soc. C. E., May, 1923, p. 840.

† *Loc. cit.*, p. 844.

Selection of Type of Curve.—Mr. Hapgood* and Mr. Hall have raised the question whether engineering data can be represented properly by the Type III frequency curve. From a purely theoretical standpoint, each problem should be studied separately, and that type of frequency curve should be used which satisfies the criterion, K, based on the actual values of the coefficients, β_1 and β_2, which are computed from the data. This can be done in actuarial and general statistical work, where the number of observations runs into the hundreds of thousands, and the coefficients can be obtained with considerable accuracy.

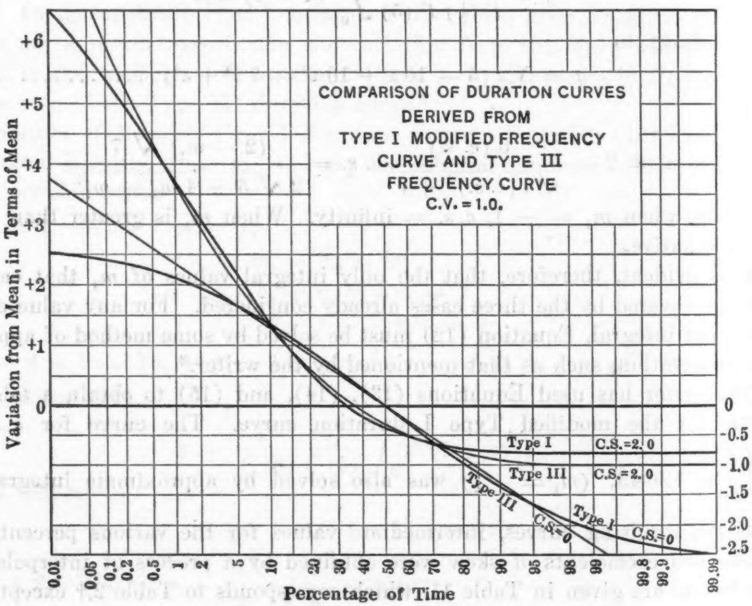

FIG. 16.

For ordinary engineering work, however, the observations are very limited in number. As a result, although the coefficient of variation can be computed with a fair degree of precision, the coefficient of skew can be determined only approximately; and the fourth moment coefficient, or μ_4, although it can be computed, will probably be worthless as far as precision is concerned.

In the case of the Type I curve, Mr. Hall has shown that the coefficient, β_2, which depends on μ_4, must lie between certain limits. He avoids the difficulties just mentioned by adopting a value for β_2, expressed in terms of β_1, which will represent average conditions. This affords a satisfactory solution of the problem, and, as has been shown, makes it possible to evaluate the frequency curve when only the coefficients of variation and of skew are known.

* *Proceedings*, Am. Soc. C. E., September, 1923, p. 1644.

TABLE 11.—DURATION CURVE FACTORS. MODIFIED TYPE I FREQUENCY CURVES.

(Skew Curve Factors to be Multiplied by Coefficient of Variation and Added to or Subtracted from Mean.)

Coefficient of skew, c. s.	Terms above mean, in percentage.	Variation from Mean, for c. v. = 1.0. Percentage of Time =																					
		0.000001	0.00001	0.0001	0.001	0.01	0.1	1.0	5	10	20	30	40	50	60	70	80	90	95	99	99.9	99.99	99.999	99.9999	99.99999	99.999999
0	50.0	+2.64	+2.63	+2.62	+2.59	+2.53	+2.39	+2.08	+1.64	+1.34	+0.92	+0.57	+0.28	0	−0.28	−0.57	−0.92	−1.34	−1.64	−2.08	−2.39	−2.53	−2.59	−2.62	−2.63	−2.64
0.2	48.4	+3.06	+3.04	+3.00	+2.94	+2.83	+2.66	+2.25	+1.72	+1.37	+0.89	+0.54	+0.23	−0.05	−0.32	−0.60	−0.93	−1.30	−1.56	−1.91	−2.12	−2.24	−2.26	−2.27	−2.28	−2.28
0.4	46.7	+3.54	+3.49	+3.44	+3.35	+3.18	+2.95	+2.42	+1.79	+1.39	+0.87	+0.50	+0.18	−0.09	−0.37	−0.63	−0.93	−1.26	−1.47	−1.75	−1.88	−1.95	−1.96	−1.97	−1.97	−1.97
0.6	45.1	+4.06	+4.00	+3.92	+3.80	+3.59	+3.24	+2.58	+1.85	+1.41	+0.85	+0.45	+0.14	−0.13	−0.41	−0.65	−0.92	−1.21	−1.38	−1.59	−1.67	−1.71	−1.71	−1.71	−1.71	−1.71
0.8	43.4	+4.62	+4.55	+4.43	+4.27	+4.00	+3.55	+2.75	+1.90	+1.42	+0.83	+0.41	+0.09	−0.17	−0.44	−0.67	−0.91	−1.15	−1.30	−1.44	−1.48	−1.49	−1.49	−1.49	−1.49	−1.49
1.0	41.8	+5.20	+5.10	+4.95	+4.75	+4.42	+3.85	+2.92	+1.95	+1.43	+0.80	+0.37	+0.05	−0.21	−0.47	−0.68	−0.89	−1.09	−1.21	−1.30	−1.32	−1.32	−1.32	−1.32	−1.32	−1.32
1.2	40.1	+5.8	+5.70	+5.50	+5.25	+4.83	+4.15	+3.09	+1.99	+1.43	+0.77	+0.33	+0.01	−0.25	−0.49	−0.69	−0.86	−1.03	−1.12	−1.17	−1.17	−1.17	−1.17	−1.17	−1.17	−1.17
1.4	38.5	+6.5	+6.35	+6.05	+5.75	+5.25	+4.45	+3.25	+2.03	+1.42	+0.73	+0.29	−0.04	−0.29	−0.50	−0.68	−0.83	−0.97	−1.03	−1.06	−1.06	−1.06	−1.06	−1.06	−1.06	−1.06
1.6	36.8	+7.2	+6.95	+6.65	+6.25	+5.67	+4.75	+3.40	+2.07	+1.41	+0.69	+0.24	−0.08	−0.32	−0.51	−0.67	−0.80	−0.91	−0.95	−0.96	−0.96	−0.96	−0.96	−0.96	−0.96	−0.96
1.8	35.2	+7.8	+7.6	+7.2	+6.75	+6.08	+5.05	+3.54	+2.10	+1.39	+0.64	+0.19	−0.12	−0.35	−0.52	−0.65	−0.76	−0.85	−0.87	−0.87	−0.87	−0.87	−0.87	−0.87	−0.87	−0.87
2.0	33.5	+8.5	+8.2	+7.8	+7.25	+6.50	+5.85	+3.67	+2.13	+1.37	+0.58	+0.13	−0.17	−0.37	−0.52	−0.63	−0.71	−0.78	−0.79	−0.80	−0.80	−0.80	−0.80	−0.80	−0.80	−0.80

This elimination of μ_4, however, does not answer the question as to what type of curve should be used to represent the data, for the type can only be determined, positively, by means of the criterion, K, and the latter must be computed by the use of the coefficient, μ_4. If the computed value of μ_4 is worthless, then the value of the criterion cannot be depended on. In general, therefore, the type of curve must be determined from other considerations. There are two characteristics of the data that may be used in this connection:

(1).—The Coefficient of Skew.—It has been shown by the writer that the *c. v.* for Type III must be less than $\frac{1}{2}$ *c. s.* Mr. Hall shows that, for the modified Type I curve, *c. v.* is less than $\frac{1}{7}\left(\sqrt{4\,(c.\,s.)^2 + 7} + 2\,c.\,s.\right)$.

Mr. Hall has plotted the equations, $c.\,v. = \frac{1}{7}\left(\sqrt{4\,(c.\,s.)^2 + 7} + 2\,c.\,s.\right)$ and $c.\,v. = \frac{1}{2}\,c.\,s.$ in Fig. 13,* and finds that practically all the points lie between these two plotted lines, indicating that the data from which these points were derived belonged to the Type I rather than the Type III curve. As the two curves are nearly parallel, it may be assumed that if the computed *c. v.* lies below the lower curve, the data belong to the Type III curve; whereas, if *c. v.* lies between the lower and the upper curve, the data belong to the Type I curve. In either case, the value of *c. s.* computed from the data should be adjusted as explained subsequently, before this test is applied.

(2).—The Upper Limit of the Duration Curve.—Theoretically, a Type I curve has a definite upper limit, whereas a Type III curve has no upper limit. Therefore, if the data show a decided tendency toward a limiting value, this tendency should be respected. For example, in Fig. 6,† the data for the Hudson River indicate a curve which is concave upward at the left end. This implies that there is no definite upper limit. The data for the Croton River show a tendency to be concave downward, which may indicate an upper limit. The theoretical curves in Fig. 16 show the same typical shapes, Type I being concave downward and Type III concave upward at the upper end.

In any particular case, the engineer will have to use his own judgment as to which type of curve to adopt. As a rule, the number of observations will be so small that no positive solution is possible. In this connection, it will be well to remember that the Type III curve will give higher values at the upper end and smaller values at the lower end. Therefore, when there is any doubt, a Type III curve would be on the safe side.

The engineer must also use his judgment in deciding whether he may assume that the data have a definite upper limit which can never be exceeded. For instance, if $c.\,s. = 1$, Table 11 shows that the maximum positive variation is 5.2 *c. v.*; or, with a *c. v.* of 0.5, the maximum possible value would be 3.6 × the mean $((5.2 \times 0.5) + 1.0 = 3.6)$. If the data are of such a nature that this value conceivably might be exceeded, then a curve of Type I is obviously unsuitable.

* *Proceedings*, Am. Soc. C. E., October, 1923, p. 1756.

† *Proceedings*, Am. Soc. C. E., May, 1923, p. 851.

TABLE 12.—DURATION CURVE FACTORS. TYPE III FREQUENCY CURVE.
(For Use in Plotting Skew Curves for Extreme Percentages-of-Time.)

Coefficient of skew, $c.\ s.$	VARIATION FROM MEAN, FOR $c.\ v. = 1.0$. PERCENTAGE OF TIME =													
	0.000001	0.00001	0.0001	0.001	0.01	0.1	1.0	99.0	99.9	99.99	99.999	99.9999	99.99999	99.999999
0	+5.60	+5.20	+4.76	+4.27	+3.73	+3.09	+2.33	−2.33	−3.09	−3.73	−4.27	−4.76	−5.20	−5.60
0.2	+6.65	+6.12	+5.48	+4.84	+4.16	+3.38	+2.48	−2.18	−2.81	−3.32	−3.77	−4.16	−4.49	−4.80
0.4	+7.73	+7.05	+6.24	+5.42	+4.60	+3.67	+2.62	−2.03	−2.54	−2.92	−3.27	−3.56	−3.80	−4.02
0.6	+8.84	+7.99	+7.02	+6.01	+5.04	+3.96	+2.77	−1.88	−2.28	−2.53	−2.77	−2.97	−3.12	−3.23
0.8	+9.98	+8.95	+7.82	+6.61	+5.48	+4.25	+2.90	−1.74	−2.03	−2.18	−2.32	−2.42	−2.48	−2.50
1.0	+11.16	+ 9.93	+ 8.63	+7.22	+5.92	+4.54	+3.03	−1.59	−1.80	−1.88	−1.93	−1.98	−1.99	−2.00
1.2	+12.36	+10.93	+ 9.45	+7.85	+6.37	+4.82	+3.15	−1.45	−1.59	−1.63	−1.65	−1.66	−1.67	−1.67
1.4	+13.59	+11.95	+10.28	+8.50	+6.82	+5.11	+3.28	−1.32	−1.40	−1.42	−1.43	−1.43	−1.43	−1.43
1.6	+14.84	+12.99	+11.12	+9.17	+7.28	+5.39	+3.40	−1.19	−1.24	−1.25	−1.25	−1.25	−1.25	−1.25
1.8	+16.12	+14.05	+11.96	+9.84	+7.75	+5.66	+3.50	−1.08	−1.11	−1.11	−1.11	−1.11	−1.11	−1.11
2.0	+17.42	+15.12	+12.81	+10.51	+8.21	+5.91	+3.60	−0.99	−1.00	−1.00	−1.00	−1.00	−1.00	−1.00

Deriving c. v. and c. s. from the Records.—Mr. Hazen* brings out the fact that, although the computed value of *c. v.* is largely independent of the number of terms in the record, the *c. s.* is decidedly affected by the length of record. In the theoretical curves under discussion, the *c. s.* is a mathematical function of the curve; and the particular values used depend on the assumption of a continuous curve. In other words, in order to obtain the given value of the *c. s.* for one of these curves, an infinite number of terms would have to be used. It is clear, then, that the value of the *c. s.* computed from a record of moderate length would have to be adjusted somewhat before it could be compared with the theoretical *c. s.* of the corresponding duration curve.

For Type III curves, Mr. Hazen suggests the correction factor, $F = 1 + \frac{8.5}{n}$, where n = number of terms in the record, and F = factor by which the computed *c. s.* must be multiplied to obtain the adjusted *c. s.* This formula agrees well with the results given by the writer in Table 3† for values of n between 10 and 100.

A similar correction factor derived from the Type I curves probably would give somewhat smaller results, since the Type I curves do not give such large values for the higher terms. The writer is unable to make the necessary calculations at the present time to obtain an exact expression for the factor for curves of Type I, but would suggest $F = 1 + \frac{6}{n}$ as giving suitable results. In any case, the factor, $1 + \frac{8.5}{n}$, would give results on the safe side.

Following out Mr. Hazen's comments on the proper nomenclature for this coefficient, the writer would suggest the following definitions: The "computed coefficient of skew" is the value for *c. s.* derived from the original data without correction. The "adjusted coefficient of skew" is the value obtained by multiplying the computed coefficient by the correction factor noted previously, and corresponds to the values for *c. s.* given in the duration curve tables (Tables 11 and 12).

Logarithmic Duration Curve Factors.—Mr. Hazen has developed an ingenious method for deriving duration curve factors on a logarithmic basis, but the writer is unable to discern any logical foundation for these factors. Mr. Hazen notes that his logarithmic factors give close agreement with the Type III factors within the limits of ordinary records, but the same is true of Type III and Type I curves. Within these limits, almost any type of duration curve could be used.

In reference to this question, Mr. Hall writes:‡

"I do not consider that these logarithmic factors are correct, after having made a careful study of the subject. * * * Mr. Hazen calls attention to the agreement for low values of *c. s.* and *c. v.* and in the center of the curves. The reason for the agreement in the first cases is that with small skewness

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1292.

† *Proceedings*, Am. Soc. C. E., May, 1923, p. 848.

‡ Letter to the writer, dated August 6, 1923, subsequent to the presentation of Mr. Hall's discussion.

and range, the error due to using the logarithmic method does not become apparent. As to the second part—close agreement in the center of the curve—this is of slight value as the principal value of the frequency curve to engineers is to determine extreme values. The Pearsonian curves cannot be improved upon, as far as I can see, and they certainly have a sound mathematical basis. The only difficulty in their application is the determination of the correct constants with the records available."

One characteristic of the logarithmic curves deserves notice. All these curves, if extended far enough at the lower end, will approach zero. This would mean that all duration curves, whatever the coefficient of skew, would have zero as a lower limit, which does not seem reasonable. Many classes of data have a more or less definite lower limit which is considerably above zero.

Mr. Hazen also points out that the high terms of the record have greater weight relatively in the determination of the *c. s.* than the low terms, and he claims that, on this account, the method will show the high terms with greater accuracy than the low terms. This condition, however, is somewhat influenced by the fact that the position of the duration curve is affected by the *c. s.* much less at the lower end than at the upper end. The examples that have been given, show an agreement with the data, which is generally as good at one end of the curve as at the other end.

Mr. Hapgood shows that, in particular cases, the computed value of the criterion does not point to any definite type of frequency curve, and suggests that each particular case be studied by the curve that theoretically applies to it. The writer has attempted to demonstrate that this is what should be expected, due to the fact that the computed value of the fourth moment coefficient cannot be depended on. The type of curve to be used must be determined by other considerations, as has already been explained.

Mr. Hall states that the application of the first test mentioned (the relation between the maximum allowable *c. v.* and the value of *c. s.*) to stream-flow data presented by Mr. Hazen and by himself indicates that nearly all the streams are best represented by the Type I curve. If the computed values of *c. s.* for these records are adjusted, as previously explained, so as to correspond with the *c. s.* for the theoretical curves, it will probably be found that more of the streams agree with the Type III curve.

Mr. Dana* suggests that a modification of the ordinary probability plotting paper be developed on which skew curves would plot as straight lines. This is partly accomplished by the logarithmic-probability paper; but any paper constructed so that a given skew curve plots on it as a straight line, would produce a curved line for a curve with a different coefficient of skew. Such a procedure would require a large variety of probability plotting papers. However, it may be that a paper constructed so as to give a straight line for the modified Type I duration curve with *c. s.* of zero would be useful for the study of data that belong to the Type I curve. Experimentation with such a form of plotting paper might be instructive. It should be remembered that any type of duration curve can be plotted on any form of probability

* *Proceedings*, Am. Soc. C. E., September, 1923, p. 1646.

paper, the only question being as to which paper shows the results to the best advantage.

Theoretical Shape of Frequency Curve.—Mr. Tucker* has raised the question whether the shape of the theoretical frequency curves agrees with the distribution of data obtained from actual experience. Although it is true that, in some cases, the data will show signs of having more than one mode, or of not becoming tangent to the base at the ends of the distribution, as a general rule, the frequency distributions are found to approximate the conditions assumed in the derivation of the general equation of the frequency curve. Moreover, even where definite irregularities exist in the frequency diagram, these irregularities are very largely absorbed by the duration curve. When the data are arranged to give a duration diagram, they are almost certain to give results approximating the general form of the theoretical duration curve.

Any discussion of the proper shape for the frequency curve has little practical significance as far as engineering data are concerned. For in nearly all engineering work, the number of observations is not large enough to permit the construction of even a very rough frequency curve, although a satisfactory duration curve can be obtained. Consequently, some assumptions must be made regarding the proper shape of the frequency curve. The assumptions adopted were based on experience with data which were sufficiently numerous to indicate the form of the curve.

It should be remembered that, in developing a mathematical method for treating statistical data, the process evolved must be reduced to the simplest form that is consistent with the requirements. If the assumptions regarding the frequency curve are too complicated, the method becomes unwieldy. The writer believes that his methods are so simple that the average engineer can handle them without difficulty.

Mr. Goodrich† presents an interesting modification of the Type III frequency curve. His method might be useful for the construction of a frequency curve that would represent the data in a general way. It should be noticed, however, that any frequency curve must have an area equal to the sum of the original data. The curve obtained by Mr. Goodrich's method does not satisfy this requirement.

Although the use of the frequency curve itself is very limited as far as most engineering work is concerned, due to the scarcity of observations, either the Type III (Equation (8)), or the Type I curve, $y = cz^{m_1}(1-z)^{m_2}$, can be plotted with very little difficulty after the constants are computed by means of the *c. v.* and *c. s.*

A detailed study of the whole subject of frequency curves involves a large amount of work, and for this reason the writer is greatly indebted to those who have taken part in the discussion. As a summary, the following procedure for the study of actual data is suggested:

1.—Arrange all observations in order of their magnitude, express the items in terms of their mean, and compute the *c. v.* and the *c. s.*

* See p. 1860.

† *Proceedings*, Am. Soc. C. E., October, 1923, p. 1759.

2.—Plot the data on arithmetic probability paper, and pass an approximate curve through the plotted points.

3.—Select the proper type of duration curve to represent the data. This is a matter of judgment, which may be guided by:

(*a*).—*The Adjusted Coefficient of Skew.*—Multiply the computed *c. s.* by $\left(1 + \frac{8.5}{n}\right)$, where n is number of items in the record, to obtain the adjusted *c. s.* If the resulting value is greater than 2 *c.v.* the data probably belong to the Type III curve; if it is less than 2 *c. v.* the data probably belong to the Type I curve, and the adjusted *c. s.* may be taken as equal to the computed *c. s* $\times \left(1 + \frac{6}{n}\right)$.

(*b*).—*The Shape of the Plotted Duration Curve.*—If the approximate curve appears to be concave upward at the higher end, it is probably in Type III. If it is concave downward at the upper end, it is probably in Type I.

If in doubt as to which type of duration curve to adopt, Type III may be selected as giving results on the safe side.

4.—Compute points for the duration curve by means of the factors in Table 2 and Table 12 for Type III or by Table 11 for Type I. In taking the factors from these tables, the adjusted value of *c. s.* should be used.

5.—Plot the theoretical duration curve on the same diagram with the record data and note how well the plotted points agree with the theoretical curve. If the agreement is reasonably close, the record may be extended upward or downward by scaling values from the theoretical curve.

The final results must not be considered as a precise forecast of future conditions, but merely as a guide to common-sense judgment. In this connection, Mr. Hazen's statement is worth repeating: "If the data show a well-marked tendency to arrange themselves in some other way than the theoretical curve, that tendency should be respected."

THE DESIGN OF EARTH DAMS

Discussion*

By Messrs. John E. Field, J. C. Stevens, and Joel D. Justin.†

John E. Field,‡ M. Am. Soc. C. E. (by letter).§—The discussions on this paper have brought out many valuable and practical hints. The paper itself takes up more of the theoretical details, which have in the past been little developed. The formulas indicate the necessity of precautionary measures, and there are numerous practical suggestions of how remedies may be provided in the final design.

One serious difficulty in making investigations for the purpose of formulating rules and processes for the use of the designer of earth dams is in the suppression of valuable data on structures of questionable safety and unsatisfactory performance and the discouragement of investigations. The owners of such structures and sometimes the engineers involved in their construction, do not wish either investigation or publicity.

The author has stressed somewhat the use of core-walls as a means of overcoming the dangers of high lines of saturation and of piping. It is difficult to oppose water or prevent it from following its natural tendencies, but it is easy to lead and direct it. A core-wall is in positive opposition to all the natural tendencies, and almost unyielding. Are there not less radical measures that will accomplish the objects desired more moderately?

The two objects are to lower the lines of saturation and to prevent piping: The natural process of lowering a water surface is to draw off the water at the bottom, and piping is prevented by plugging the upper and not the lower end or middle of the pipes. As remedies, there are drains in the one case, and blankets of impervious material over the dam and reservoir bottoms in the other.

A satisfactory example of drains is seen in the combination earth and rock-fill dam, although various modifications such as rock toes, rock drains (both vertical and horizontal), and tile drains, are often sufficient.

Charles H. Paul, M. Am. Soc. C. E., has mentioned|| the blanketing of reservoir bottoms. Many dams have an impervious section of earth at or near the upper face, to prevent piping.

Mentioning only structures in Colorado which have come under the writer's observation, the following have been successful:

* Discussion on the paper by Joel D. Justin, M. Am. Soc. C. E., continued from October, 1923, *Proceedings*.

† Author's closure.

‡ Denver, Colo.

§ Received by the Secretary, September 19, 1923.

|| *Proceedings*, Am. Soc. C. E., August, 1923, p. 1321.

The Rio Grande Reservoir Dam.—This is a combination rock-fill and earth dam, with hand-laid rip-rap.

The Terrace Reservoir Dam.—This is of the hydraulic-fill type where the natural channel was filled with sediment from the river and from the waste water after sluicing.

The Santa Maria Dam.—This is also built by hydraulic-sluicing, with hand-laid rip-rap and a heavy gravel toe.

In each of these three dams there is some seepage, but no moisture on the outer slopes. The toe, which is of heavy, coarse materials, prevents washing and sliding. There is no core-wall.

In the Terrace Dam a core-wall was built. It did not reach impervious material and several holes were punched in it; also, a notch was blasted in its top edge. It acts not as a core, but as a baffle-wall.

Most of the reservoir dams on Pike's Peak are of disintegrated granite, as is also the Worster Dam on the Cache La Poudre River. These dams seep badly, and their lower slopes are saturated, but the coarse material drains and does not wash. On the Worster Dam, a heavy toe of large rock and backing prevents slipping. Perforated pipes driven horizontally into the dam were inserted as an added precaution.

Jackson, Barr, Riverside, Prewett, and Empire Reservoir Dams are of very light sandy material and are located in a sand dune country. There is great loss from seepage, so much so that the value of the reservoirs as storage basins is reduced and drainage ditches have been constructed to reclaim the lost water. The first four of the dams mentioned have concrete facings. The losses, as in the case of reservoirs on Pike's Peak, are principally through the bottoms of the basins. The writer has recommended as a remedy the sluicing of material into the shallow water of the shore and particularly on, and in the vicinity of, the embankments.

The dam failures or partial failures in this locality are as follows:

Marshall Lake.—A dam of adobe material on a boggy foundation and without drain or core. It has had several slips.

Standley Lake.—Presumably this is of early clay, uniform throughout, without drain or core. The compacting was irregular, and often entirely neglected. In this dam, the lines of saturation are very erratic: Some start at the water surface, curving rapidly downward and then upward to nearly the water-surface elevation, and reach the outer slope in some cases below the ground surface and in others well up on the slope.

The Horse Creek Reservoir.—This dam failed through water passing beneath the fill and over the shallow bed-rock of shale. The basin above the dam had been stripped to make the fill. This gave the water full opportunity to reach bed-rock and flow into cracks or crevices. Added thickness over the bed-rock (as a blanket) and baffle-walls should have been provided.

The Apishapa Reservoir.—This dam, which failed on August 23, 1923,* was provided with an ample blanket up stream, about 19 ft. in depth, the result of the accumulation of silt, and with ample cut-off walls in the base

* *Engineering News-Record*, August 30, 1923, p. 357 and September 13, 1923, p. 418.

and on the sides of the canyon. The material was poor, containing more than 6% of solubles, was practically uniform from front to back, and was insufficiently compacted. Puddling in the center as in hydraulic fills might have prevented the failure, inasmuch as the contained salts were readily soluble and the lavish use of water in settling would have removed most of them. The use of a concrete core would have resulted in a saturated condition of the material above the core and probably in the slipping of the upper slope. A combination earth and rock-fill dam is indicated in this case. The direct cause of this failure was that the fill had dried out, and that settlement within the dam, not visible on the surface, had occurred.

Investigations of existing structures, if carefully and critically studied, will be valuable, provided they include a knowledge of the material, of the method of construction, and of the adequacy of inspection.

Castlewood Dam, originally a masonry retaining wall backed with loose rock-fill, did not reach impervious material. Piping caused a washout beneath the structure on two occasions. An earth blanket reaching from near the top of the wall, on a 4 to 1 slope, was put in, and no trouble from seepage or piping has occurred since, a period of more than twenty years.

The Beaver Park Dam has a concrete face backed with loose rock, and allowed 130 cu. ft. of water per sec. to escape through the bed-rock formation on one side and the bottom. The west abutment, or natural fill, proved to be of lenticular sand, gravel, and clay. The rock was tightened by drilling holes and forcing grout in the crevices and voids. This abutment was drained by a tunnel from which pipes were driven radially. Both measures were quite successful, although not completed as contemplated in the plans for the repairs.

Of three reservoirs near Pueblo, Colo., similar in every way, the only one that gave trouble had a timber diaphragm, most carefully (and expensively) constructed, extending from the crest to a considerable distance below the ground line. The upper face slipped into the reservoir, covering the outlet tubes, and the lower face was saturated for about one-fourth its height. The material was of adobe rolled and compacted by water along the diaphragm. As a substitute for this diaphragm, a gravel drain, placed vertically under the crest and possibly extending to the crest, would have been a more profitable precautionary measure. However, from the performance of the other two reservoirs, the omission of the core-wall and the placing of a gravel or rock toe are indicated as the proper design.

Going farther afield, a report with plans of a dam in California has been submitted to the writer, in which the construction is in part combination rock-fill and earth, and in part all earth, with a reinforced concrete core-wall extending throughout to impervious material or bed-rock. It is reported that the core has moved or slipped, and that seepage flows out under the rock-fill and also has appeared on the toe of the all-earth section. The core-wall is probably broken. Its presence in the rock-fill and earth section is a mystery, and its utility in the earth section is questionable. In this dam, the omission

of the core seems proper and the placing of a rock toe on the all-earth section is advisable.

The failure of the Lyman Dam in Arizona was due to its having been filled too rapidly after having been dried out. Some soluble salts were present in the section where the first break occurred.

For some years, the writer has been of the opinion (1) that, generally, core-walls should be avoided; (2) that where the bed-rock is shallow, baffle-walls are sufficient; (3) that trenches backfilled with selected material are preferable to sheet-piling; and (4) that drains are practical in making dams more secure from sloughing.

As to the core-wall, one objection is that the embankment is divided into two parts, one a saturated mass above the core, which, due to its liability to slip, must have a flatter slope, and the other a comparatively dry mass below the core, giving unequal pressures on it, especially in those cases where the reservoir is frequently filled and emptied.

The upward movement of water on the lower side of a core is shown in Figs. 6 and 7,* of the paper by J. B. T. Colman, Assoc. M. Am. Soc. C. E., entitled "The Action of Water Under Dams." The equal pressure lines at the lower edge of the sheet-piling rise almost vertically. If, in those investigations, the area of water and sand above the model dam had been many times larger, as is the case in a reservoir, the pressures from the surface of the sand downward to the lower edge of the sheet-piling would have increased with the head.

The writer notes the statement† of Thomas H. Wiggin, M. Am. Soc. C. E.: "Mr. Merriman has described experiments in the earth dikes of Ashokan Reservoir showing no line of saturation even on the water side of the concrete core-wall after a lapse of years with water in the reservoir." The material used in the fill must have been quite unusual. This is probably a case where a core-wall was unnecessary, the upper face of the embankment being apparently as impervious as some concrete.

The saturated upper slope gives full hydrostatic pressure against the core, and water following along the contact between earth and core seeks out any weak spot. Full hydrostatic pressure exists at the lower edge of a core, and will cause seepage beneath it unless the core is carried down to impervious material. The water will then rise along the line of contact on the lower side of the core until the head is sufficient to force it laterally.

In the case of timber cores, the unequal pressures will probably open up the joints and permit the water (concentrated both as to volume and power at the open joint) to enter the lower part of the dam.

Referring to Figs. 6 to 9,‡ it appears that the lines of saturation above the core-walls would be horizontal and of the same elevation as the water surface, otherwise, either (1) there is a more ready escape beneath the lower edge of the core than seepage through the upper part of the fill can supply,

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), pp. 438–439.

† *Proceedings*, Am. Soc. C. E., August, 1923, p. 1314.

‡ *Proceedings*, Am. Soc. C. E., May, 1923, pp. 864–865.

or (2) the water is following the line of contact between the core and fill, or (3) the core is ruptured and leaking.

Lines of saturation are generally shown as straight lines from the water surface on the inner slope to the point where seepage appears on the lower slope. The writer recently had occasion to investigate a dam and canal, with results as given in Fig. 53. From this and other observations made by him, the underground water surface appears to be concave, the concavity depending on the topography and material below the dam or canal. The very steep line shown in the case of the canal was due to the impregnation of its sides and bottom with fine material filtered out of the water seeping from the canal. Near the reservoir, the same cause was not so apparent. The soil through which the canal was built, and of which the reservoir embankment was composed, was a light, sandy loam, the height of the water-table being quickly affected by a rise or fall in canal or reservoir.

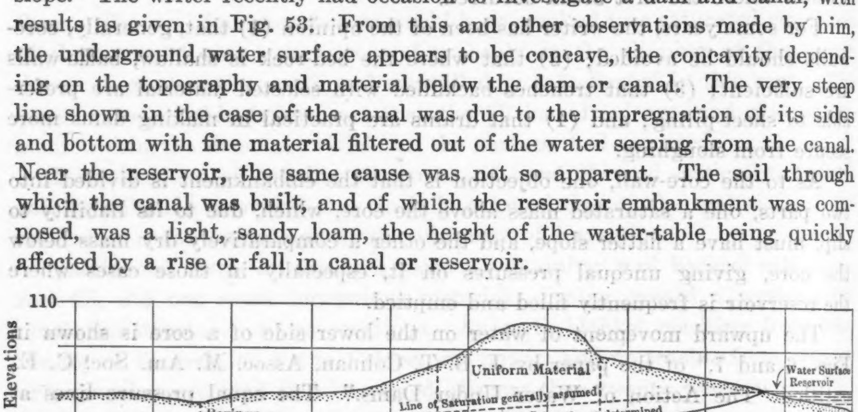

FIG. 53.

The predetermination of probable lines of saturation is desirable, but whether determined by tests of material in a formula as given by Mr. Justin, or by models constructed to scale, the results are not sufficiently dependable to justify the exclusion of precautionary measures, such as (*a*) blanketing the reservoir bottom and river channel, (*b*) using baffle-walls or trenches, and, particularly, (*c*) providing drainage. This drainage may be provided by a rock-fill in a part or the whole of the lower section of the dam, by drains of rock or tile, or by the selection of the most porous material for the lower toe and of the most impervious for the upper face. Either a gradual transition, or a radical change as in the case of rock-fill for the lower part, may be adopted.

Mr. Paul, in his discussion, states, "The writer is glad to note that the author contradicts the old textbook theory that a dam should not be built unless impervious material is available." One may reasonably add that the use of some porous material is as essential as impervious material to proper construction of earth dams.

In Fig. 12,* the core-wall is shown at the water surface. In this position, seepage follows the surface of contact more readily. When the water is raised and lowered, wave action will expose more and more of the wall—unless rip-rap is provided—and the upper part must sustain the earth pressure behind as a retaining wall. The placing of the core at the water surface would seem to involve some danger and expense, and the advantage gained by the fact that a greater part of the fill lies behind the core than in other construction is somewhat offset by disadvantages.

On large reservoirs, there is generally a watchman or gate-keeper, and a road passes over the dam. This may be the reason the writer has never seen a large earth dam or an earth dam on a large reservoir attacked by boring animals, although Colorado and other Western States enjoy their full quota of muskrats, gophers, and badgers. The danger from boring animals seems remote on large reservoirs.

The use of concrete for facing dams is quite common in the West. It was claimed that it would aid in preventing seepage; of the many dams observed by the writer, however, the concrete facing has had little effect, the line of saturation being the same as on similar dams without facing. The approved present practice in Colorado is to cast the slabs in place, 30 to 50 ft. long horizontally, 6 in. thick, reinforced with expanded metal, dovetailed at the joints, and caulked with roofing material and asphaltum. Under the joints a broad beam is sometimes used, which serves as a surface on which the slabs may slide. Weep-holes are provided. Because waves run high on the smooth concrete, provision should be made to break their continuity by triangular projections on the surface. The failure of the Bootleg Reservoir indicated clearly that where water recedes rapidly, the hydrostatic pressure behind the slabs will cause rupture and slipping.

In the case of the Empire Dam, previously mentioned, the seepage is greater than in the Riverside Dam, indicating that in sand-fills the concrete facing decreases seepage.

In nearly all cases, especially on rock rip-rap, a bed of gravel is necessary. The Fossil Creek Reservoir was beautifully paved with a hard sandstone which occurred in layers of from 6 in. to more than 1 ft. in thickness. The rock was placed on edge, with close joints, and the crevices were filled with spalls and gravel; nevertheless, the wave action sucked out the dirt beneath the rip-rap and caused large areas to fail. In the repairs, a gravel base was used with satisfactory results.

Competent and experienced engineering is essential in the construction of earth dams, and a thorough apprenticeship in this class of work should precede attempts either to conduct the investigations or to supervise construction. Given thorough investigation and reliable data, it is probable that the designs of experienced engineers would differ only in details. In their order of importance, design, in the writer's opinion, would rank third, as compared with construction and investigation.

* *Proceedings*, Am. Soc. C. E., May, 1923, p. 866.

J. C. STEVENS,* M. AM. SOC. C. E. (by letter).†—The author has failed to mention in his six criteria, one of the most important factors affecting the safety of earth dams, that is, the matter of foundations. It is often necessary to build earth dams on alluvial deposits, which are seldom as compact as the prism of the dam itself, with the result that troublesome settlement of the embankment often occurs.

The several methods of dam construction, whether by rolled layers, hydraulic-fill, or a combination of these methods, generally produces an embankment that is much more compact than the original soil on which it is founded. Subsequent settlement is due to a greater compacting of the foundations, rather than of the fill itself, and frequently results in uplift of the soil above and below the dam.

The author has shown that the foundations will unavoidably become wet, often completely saturated, although the original soil may have been dry before construction. Railroad embankments and dikes built across swampy lands often disappear almost completely, and have to be rebuilt several times before a stable condition is reached. In certain kinds of swampy soils, it is practically impossible to maintain a fill of any appreciable height.

The same conditions are faced when building earth dams on alluvial soils; the foundations may be dry and appear to be firm, but on being saturated from water behind the dam, they become swampy in character and may constitute a real menace unless especial precautions are taken.

Such alluvial deposits are rarely uniform in texture. No amount of sampling and testing will give reliable data from which the line of saturation may be even approximated by computation. The only practicable remedy lies in intelligent drainage of the site prior to construction.

The writer was in charge of the design and construction of seven earth dams in connection with the Serós hydro-electric development in Spain in 1913-14.‡ All these dams were built on alluvial foundations in dry gulches, in layers, sprinkled and rolled. The up-stream part was of selected clayey material, which became very compact. The middle part consisted of borrow-run, was spread in thicker layers, and also sprinkled and rolled. The down-stream part was of coarser material, dense, but porous. The object was to build a prism, the porosity of which increased down stream. Fig. 54 shows a cross-section of Dam No. 3.

The foundations of these dams were not specially drained, and settlement occurred on all of them. The greatest settlement occurred on Dam No. 3, and Fig. 55 shows the amount progressively for 9 months, after the reservoir was first filled. Whether the dam has reached a condition of stability since that time, the writer does not know.

Little or no water percolated through the dam itself, most of it coming through the original hillsides and saturating the foundations at the foot of the slopes. It will be noted that the settlement is greatest over these points. The total settlement at the foot of the right slope was nearly 1 m. in 9 months.

* Cons. Hydr. Engr. (Stevens & Koon), Portland, Ore.

† Received by the Secretary, September 26, 1923.

‡ *Engineering-News*, September 3 and 10, 1914; also, *Engineering Record*, August 29 and September 5, 1914.

The total quantity of visible seepage from this dam never exceeds 1 sec-ft. Although the safety of the structure has never been questioned, considerable additional material had to be added from time to time to maintain the required freeboard.

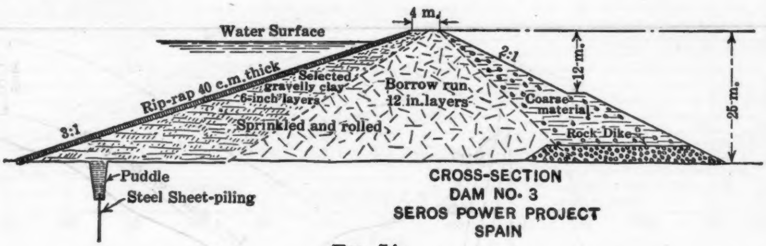

FIG. 54.

Another dam recently designed and built for the City of Coquille, Ore., under the direction of the writer's firm, is of more than passing interest. The foundations were of clay, and one abutment was on the site of an ancient slide. Fir trees 3 ft. and more in diameter had grown on its surface after the slide had occurred. Springs developed in the foundation, which became very soft

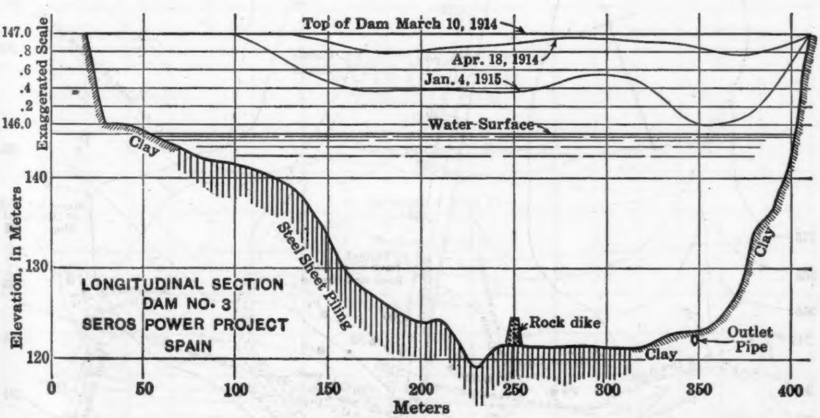

FIG. 55.

and yielding wherever it was saturated. An extensive drainage system was built in the foundations and abutments before placing the fill. Both abutments were drained by tunnels in which was placed a 6-in. tile, and then each was back-filled with gravel and broken stone. These abutments are shown in Fig. 56.

After the completion of the dam, six wells were driven into the fill by which the saturation and pressure lines could be determined. These wells were of 2-in. pipe with well points. Three were perforated to within 10 ft. of the surface of the dam and are called saturation wells; three are water-tight, except at the bottom, are driven below the stripping line, and are called pressure wells. Their location in plan is shown in Fig. 56 and in section in Fig. 57.

Fig. 58 shows the elevation of water in them, and also in the reservoir during the first filling. No water has appeared to date in Saturation Well S_3. Fig. 57 shows the saturation and pressure lines as determined from these wells on two representative dates.

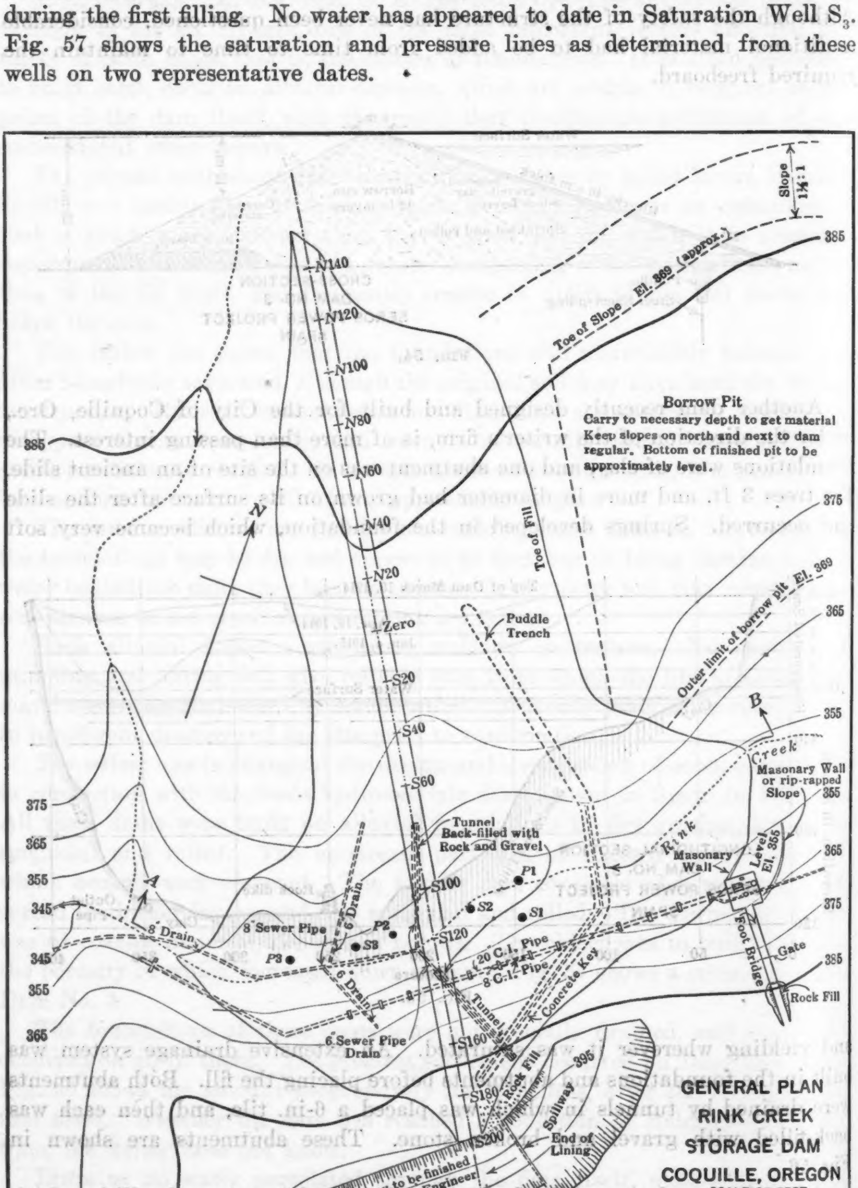

FIG. 56.

The effect of drainage is apparent in these lines. To date, no appreciable settlement of the fill has taken place. The puddle core was put in to prevent

any direct seepage along stratifications of the alluvial deposits and undoubtedly is very effective for this purpose.

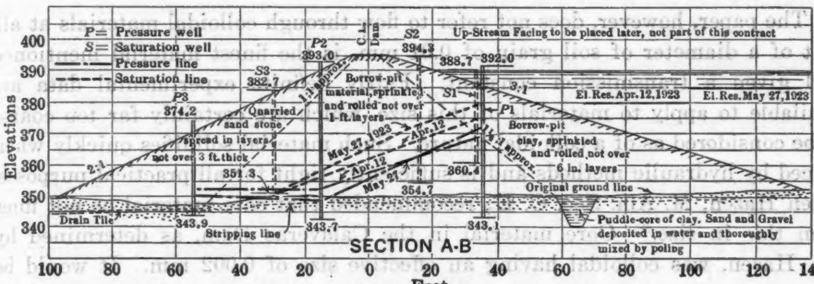

FIG. 57.

Visible seepage from the dam is negligible and has not exceeded 40 gal. per min. All of it comes from the drains constructed for the purpose in the foundations of the dam.

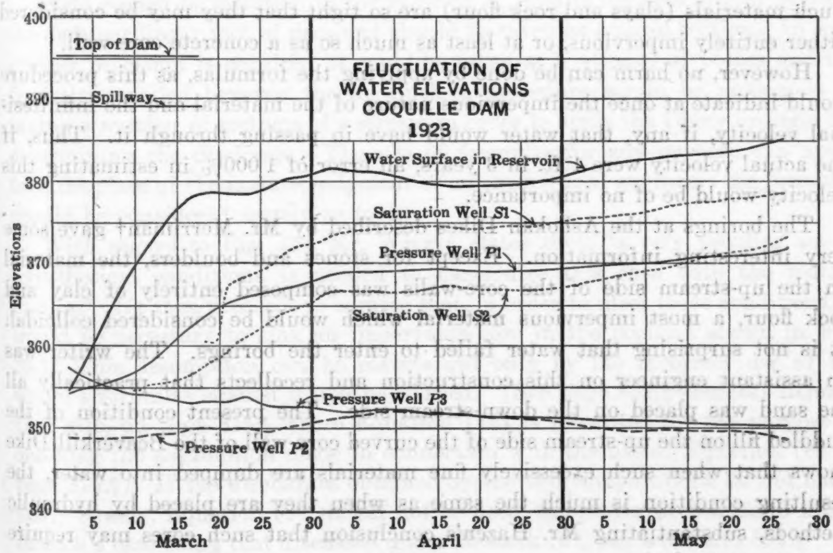

FIG. 58.

From a study of Fig. 57, noting that half the material in this dam is dry when the reservoir is full, there is a feeling of security that fully justifies the relatively small additional expense of the drainage system.

JOEL D. JUSTIN,* M. AM. SOC. C. E. (by letter).†—The author heartily agrees with the opinion of several of those who have discussed the paper that there is

* Care, The Power Corporation of New York, Watertown, N. Y.

† Received by the Secretary, September 11, 1923.

no justification for extending the application of laws experimentally determined for the flow of water through granular materials to flow through colloidal materials.

The paper, however, does not refer to flow through colloidal materials at all. Silt of a diameter of soil grain of 0.01 mm. is the finest material mentioned and given a transmission constant. Fairly definite experimental data are available to apply to materials of this size, which are certainly far too coarse to be considered as of a colloidal nature. Such material solidifies quickly when placed by hydraulic methods and is sufficiently tight for all practical purposes. Allen Hazen, M. Am. Soc. C. E., recommended that core material be not finer than this in size.* Core material in the Calaveras Dam, as determined by Mr. Hazen, was colloidal having an effective size of 0.002 mm. It would be difficult to determine just where the dividing line is, but it is somewhere between these two values. Even then, all materials finer than this critical value would not exhibit the characteristics of colloidal material. However, it can be stated positively that no materials coarser than 0.01 mm. would have any colloidal qualities. Mr. Hazen computed that, assuming the same laws of flow to apply to colloidal materials as to coarse materials, it would take 2.5 years for water to move 1 ft. through the Calaveras core on a 10% grade. Such materials (clays and rock flour) are so tight that they may be considered either entirely impervious, or at least as much so as a concrete core-wall.

However, no harm can be done by applying the formulas, as this procedure would indicate at once the impervious nature of the material and the infinitesimal velocity, if any, that water would have in passing through it. Thus, if the actual velocity were 1 ft. in 3 years, an error of 1 000% in estimating this velocity would be of no importance.

The borings at the Ashokan Dikes described by Mr. Merriman† gave some very interesting information. Except for stones and boulders, the material on the up-stream side of the core-walls was composed entirely of clay and rock flour, a most impervious material which would be considered colloidal. It is not surprising that water failed to enter the borings. The writer was an assistant engineer on this construction and recollects that practically all the sand was placed on the down-stream side. The present condition of the puddled fill on the up-stream side of the curved core-wall of the Beaverkill Dike shows that when such excessively fine materials are dumped into water, the resulting condition is much the same as when they are placed by hydraulic methods, substantiating Mr. Hazen's conclusion that such cores may require many years to drain.

With regard to Mr. Merriman's criticism‡ of Figs. 16 and 17,§ the writer would state that these cases were intentionally selected as two extremes in order to illustrate the influence of the foundation soil on the position of the line of saturation.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 1713.

† *Proceedings*, Am. Soc. C. E., October, 1923, p. 1744.

‡ *Loc. cit.*, p. 1746.

§ *Loc. cit.*, May, 1923, pp. 880 and 882.

The writer assures Mr. Wiggin that he appreciates the practical difficulties of determining representative sizes of earth particles and the depth to rock. These difficulties were discussed in detail.* On projects costing $1 000 000, it is not excessive nor unusual to spend $10 000 or $20 000 for the sub-surface investigations. If these investigations are conducted for the purpose of securing the data required for the design of an earth dam by the methods outlined in the paper, they will give sufficient information to permit the use of these methods with considerable assurance. However, no great degree of precision is expected or required with a material as inexpensive as earth.

The writer believes that Mr. Wiggin is incorrect in his statement† that the value of S_3 varies from zero to P and back again to zero. In the writer's formula, S_3 is intended to represent "the effective area of flow per linear foot of cross-section of the water flowing through the dam itself." It is probable that the upper elements of flow are about parallel to the line of saturation and the deeper ones, more nearly horizontal. The writer agrees with Mr. Wiggin that the use of $S_3 = \frac{P_3}{2}$ is somewhat arbitrary and, under Remark‡ (6), he discusses cases where such an assumption might lead to serious error, and suggests methods by which this might be obviated.

With regard to variation in the value of S_1, this is the area of flow beneath the dam per linear foot of dam and should be taken as a mean section. In case greater precision is required, it could be taken at a number of points and the formulas applied between them. Such a procedure would be analogous to taking a number of sections of a stream in order to apply the Chezy formula for determining the discharge. The use of formulas for any purpose can never take the place of the mature judgment of an experienced engineer; the formulas should be considered merely as tools.

With regard to Fig. 15§ Mr. Wiggin states that, "a very peculiar set of hypothetical conditions is needed to harmonize this case with the mathematics." He then proceeds to assume that, in order to meet these conditions, it would be necessary to sink an impervious core in the position shown in Fig. 35.‖ This assumption can be maintained only by the incorrect use of the writer's formulas against which he warned. (See page 892,¶ General Remark (6); also, on page 876,¶ where Equation (12) is presented for the purpose of checking the assumption made in connection with the use of Equation (8) on page 875.¶) If the discharges obtained by the two computations do not check fairly closely, Equation (8) should not be used, the proper procedure to follow in such a case being that discussed under Remark (6), pages 892 and 893.¶

Instead of being guided by the assumption shown in Mr. Wiggin's Fig. 35, the conditions which the writer assumed in his check calculation on the problem represented by Fig. 15 are exactly the opposite, as shown in Fig. 59. This is

* *Proceedings*, Am. Soc. C. E., May, 1923, pp. 872–873, 892–893.

† *Loc. cit.*, August, 1923, p. 1314.

‡ *Loc. cit.*, May, 1923, p. 892.

§ *Loc. cit.*, p. 877.

‖ *Loc. cit.*, August, 1923, p. 1315.

¶ *Loc. cit.*, May, 1923.

the assumption of Equation (12), which equation would also be the proper one to use in case the dam were built of absolutely impervious material on a more or less pervious foundation, for by its use it is assumed that all the discharge enters the foundation soil at the up-stream toe. The distance, h, in Equation (12), is measured from the intersection of the up-stream face with the head-water, whereas the shortest path for the water flowing under the dam starts at the up-stream toe of the dam, thus introducing an additional factor of safety. If the foundation material had been extremely pervious, as compared with the material in the dam, Equation (12) would have shown a much greater, q, than that obtained by using Equation (8),* and would thus indicate that Equation (8) should not be used. Hence, the writer can see no necessity for Mr. Wiggin's "peculiar set of hypothetical conditions."

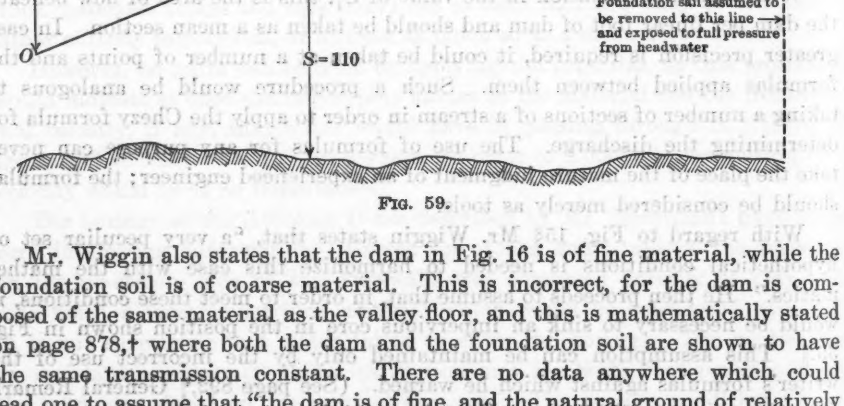

FIG. 59.

Mr. Wiggin also states that the dam in Fig. 16 is of fine material, while the foundation soil is of coarse material. This is incorrect, for the dam is composed of the same material as the valley floor, and this is mathematically stated on page 878,† where both the dam and the foundation soil are shown to have the same transmission constant. There are no data anywhere which could lead one to assume that "the dam is of fine, and the natural ground of relatively coarse, material."

In regard to the limitations of the equation, $q_1 = q_2 + q_3$, these limitations were discussed‡ by the writer and conditions were stated under which the assumption would be untenable, in which case the determination of the dimensions of the dam was to be by other methods, particularly Equation (26).§

In discussing Fig. 20,‖ Mr. Wiggin states that,

"The theory of the influence of cores on the line of saturation * * * has also an assumption which seems to be untenable, that is, that the hydraulic

* *Proceedings*, Am. Soc. C. E., May, 1923, p. 878.

† *Loc. cit.*, May, 1923.

‡ *Loc. cit.*, p. 874.

§ *Loc. cit.*, p. 906.

‖ *Loc. cit.*, p. 885.

slope through the orifice between the rock and the under side of the core-wall is $\frac{P_3}{h_3}$, or the same as the average slope from reservoir surface to toe of dam." P_3, as defined by the writer,* is the difference in elevation between head-water and the base of the dam, but h_3, instead of being the distance from head-water to toe of dam, as Mr. Wiggin states, is the horizontal distance from the point where the head-water intersects the up-stream slope to the point where the line of saturation intersects the base. h_3 is the unknown quantity which must be computed. In two of the cases, it was found that h_3 exceeded the distance to the toe, the line of saturation falling outside the toe, while in the third case the computation showed it to fall well inside. The writer agrees with Mr. Wiggin that the loss of head would be concentrated largely at the core-wall and the computations show this. It will be noted that the line of saturation takes a drop in each case. The writer also called attention to the similarity of this case to that of a partly closed valve. However, it is scarcely logical to assume, as Mr. Wiggin does, that throttling a valve will produce a greater flow.

In the case of embankments, where the up-stream part is composed of fine material and the down-stream part of coarse material, Mr. Wiggin suggests that water might rise from the foundation soil into the down-stream porous part of the dam. Under certain conditions, this would undoubtedly occur, as, for instance, in an extreme case where the up-stream part was absolutely impervious, the down-stream part and the foundation being fairly pervious. The computations for locating the position of the line of saturation in such a case would be similar to those relating to Fig. 20, and Equation (12) should be used.

In criticizing the writer's method of determining the necessary thickness of base in order to prevent piping, Mr. Wiggin states that in a leaky embankment the flow would be down and the force of gravity would act to displace the fine material, thinking, quite possibly, of flow entering the embankment from the up-stream side. When piping has progressed so far that the fine material from the up-stream end of the dam is passing through the dam and out on the down-stream side, it would be time to "take to the hills".

It is the flowing out of the water on the down-stream side of the dam that was discussed in the paper. If the piping were in the foundation soil, the flow at the exit would be vertically upward, and the theory of the formula would apply directly; if on the side of the embankment above the toe, the flow at the exit might be nearly horizontal, in which case some of the fine particles might be displaced by sliding instead of by jet action. Even in this case, the sliding would be resisted by the weight of the particle, rather than aided by it, as Mr. Wiggin states. He would scarcely say that the sliding of a masonry dam would be aided by the weight of the masonry, yet such a statement would be quite analogous. In order to guard against the uncertainties of the theory, the writer sounded a note of warning against the use of too high a velocity in the formulas, suggesting for general use in Equation (26) a permissible velocity of 0.5 ft. per min., which is one-fourth that required to move the finest silt.

Mr. Williams† emphasizes the importance of draining the down-stream part of earth dams, utilizing ditches and tile drain for this purpose. He also favors

* *Proceedings*, Am. Soc. C. E., May, 1923, p. 874.

† *Loc. cit.*, August, 1923, p. 1320.

the use, in some cases, of a lower section of dike, which in times of excessively rare floods may be washed away without causing much damage or great expense for replacement. Such a device should be widely used; it would often avoid much of the expense in meeting spillway requirements.

Mr. Paul* emphasizes the desirability of blanketing in certain cases, with which practice the writer thoroughly agrees. There are many cases in which expensive core-walls have been used, where the same or even a greater reduction in seepage could have been obtained by blanketing the face and the reservoir bottom for some distance up stream. A core-wall is often added to the design of an earth dam without a proper appreciation of its effect on the cost of the entire structure. The writer knows of a number of instances where the cost of the core-wall has exceeded the cost of the remainder of the earth dam.

Mr. Sanborn† calls attention to the use of a "gunite" membrane as a substitute for a core-wall of concrete. A number of such membranes have been used in earth dams. Many of them have been erected in a vertical position supported on timbers in the form of a fence, and others have been laid on a slope, as illustrated in Fig. 46‡ of Mr. Noetzli's discussion. As compared with a reinforced concrete core-wall of the diaphragm type, about 12 in. thick, they cost only one-third to one-half as much. The only objection to such membranes is that they are not as "fool-proof" as concrete and require greater care in construction.

The writer thanks Mr. Gourley§ for calling his attention to the error in including the Dale Dike in the list of successful earth dams as given in Table 7.‖ It will be omitted from the final list. It is pleasing to note that Mr. Gourley has included observation tubes in the Taf Fechan Dam in South Wales, in order that after the dam has been placed in service data may be obtained as to the position of the line of saturation. It is to be hoped that the data available as to the nature of the material in the dam and the foundation will also be so precise that the observations on the line of saturation will be of maximum value.

Mr. Noetzli describes and illustrates (Fig. 46), a rather unique form of diaphragm core-wall, which should prove effective and desirable under certain conditions. The writer believes, however, that a vertical diaphragm placed as in Figs. 12¶ and 22** will perform the same function as the diaphragm in the position shown by Mr. Noetzli and at less expense in most cases. In most cases, the asphalt layer shown in Fig. 46 could be safely omitted. It is included apparently for the purpose of preserving the imperviousness of the inclined diaphragm after the inevitable cracking and distortion due to settlement. If the materials on each side of a vertical diaphragm are placed carefully, cracks resulting from unequal pressure on the wall will not be extensive enough to cause much leakage.

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1321.

† *Loc. cit.*, p. 1296.

‡ *Loc. cit.*, September, 1923, p. 1618.

§ *Loc. cit.*, August, 1923, p. 1325.

‖ *Loc. cit.*, May, 1923, p. 913.

¶ *Loc. cit.*, p. 866.

** *Loc. cit.*, p. 898.

Mr. Noetzli asks: "How much is the factor of safety of an earth-fill dam?" In the case of masonry dams, factors of safety against various modes of failure have been established, but it will probably be some time before this question can be answered with the same definiteness with regard to earth dams. Under the writer's Criterion 2, it is not possible at present to define the point where the line of saturation should cut the base. It certainly would not be proper to state that the line of saturation should fall within the middle-third, for there are too many successful dams where it comes far outside this point. The safe position of the line of saturation will vary with the material. Generally speaking, the finer the material, the greater should be the distance from the down-stream toe to the point where the line of saturation cuts the base. The weight of material above is also an important factor. The line of saturation should fall well within the down-stream toe; how far inside can only be determined after a study of each particular case. With regard to Criterion 5, the writer indicated a factor of safety when, in connection with the selection of a minimum base width for safety against piping, he suggested the use of a permissible velocity of flow, through or under the base, of 0.5 ft. per min., in the usual case, which is one-fourth the velocity required to move the finest silt.*

Mr. Saville† mentions the sliding of natural hillsides when, due to the construction of a dam, water is raised against their slope. There is a great difference in the natural slopes of dry and saturated materials. When the Wissota Dam was first built, there were some steep, heavily wooded, natural slopes, about 1 on 2½ to 1 on 3, along the shore of the reservoir. The banks rose 40 to 50 ft. above head-water elevation. The underlying material was glacial drift consisting of sand with some gravel. When the water was raised, these slopes which undoubtedly had remained stable for a great many years, slid down into the water, trees and all.

In another case, the writer had permitted soil used in surfacing the down-stream face of an earth dam, to be dumped on the top of the dam. The water had been raised to head-water elevation. The soil was a heavy loam, and the dam itself was composed of a very fine sand. The contractor built a trestle part way across the dam and dumped the top soil from the trestle. At the point in question, it covered the entire top width of the dam to a depth of about 15 ft., with side slopes of about 1 on 1. At the time, it did not occur to any one that there was any danger involved in this procedure. Finally, however, the writer became worried over the great superimposed load and, on examining the up-stream face, found a number of longitudinal cracks opening up just above the water line, thus indicating impending sliding, due to the great load above. The top soil was pulled down to its final position, and no further trouble was experienced.

Mr. Saville mentions how water finds its way around the ends of earth dams, and how this condition should be investigated. Equation (26), in particular, is generally applicable. It was in connection with just such a case that the writer started the present study. On the construction of an earth dam, the

* *Proceedings*, Am. Soc. C. E., May, 1923, p. 905.
† *Loc. cit.*, September, 1923, p. 1608.

material of the side-hills, into which the dam abutted, proved to be much more pervious than had been anticipated. The writer made a study of the published data that might be useful in a case of this kind, particularly of the work of the late Frederic P. Stearns, Past-President, Am. Soc. C. E., at Wachusett and of Mr. Saville at Gatun. This study was an important factor in the decision as to the proper distance to carry the core-walls into the hillsides.

The data given by Mr. Jarvis* on accidents to earth dams will be of great value, in preventing others from making the same mistakes. His Example No. 3 illustrates very forcibly that there is a great difference in the safe slope of any given material under water and in air. The old dam at the Sevier Bridge site stood successfully with a down-stream slope of 1 on 1½, but as soon as the lower part of this slope was placed under water, a slide developed.

Mr. Holmes† refers to the case where the foundation soil is composed of layers of different materials varying greatly in their effective size and porosity. The writer agrees that the seepage would be very different from that through the same materials mixed together, in which case he would not suggest the use of the average effective size and porosity for the entire cross-section, but would apply the formulas to each layer separately and determine the seepage through each. The problem suggested by Mr. Holmes might have many variations, as, for instance, if the coarse layer were at the top. Conditions would then be very different from the case of a coarse layer at the bottom and a relatively fine layer on top, perhaps blanketing the coarser layer for a mile or more up stream. In any variations of this problem, the writer believes that the formulas will prove to be useful in predicting both the position of the line of saturation and the quantity of seepage to be expected under the conditions as determined by sub-surface investigations. The writer does not propose to "assume" any effective size, but rather to determine effective sizes and porosity experimentally by means of borings and test pits.

It is believed that the value of the conclusions reached under the methods discussed will depend on the thoroughness and accuracy of the field investigations and the soundness of the judgment exercised by the engineer. In the cases presented by the writer, in order to promote clarity, relatively simple conditions were assumed to exist, although in any actual case the conditions would seldom be so simple.

Mr. Holmes refers to the decrease in seepage with increase in compactness. The formulas take this into account, for compactness varies with porosity.‡ However, the variation in porosity in a dam of homogeneous material, due to the increasing superimposed weights at the lower depths, would not make nearly as much difference in seepage as Mr. Holmes appears to think; if considered desirable, it could be determined and used in the computations. The writer believes that this would seldom be necessary, because any additional compacting due to weight of material above will merely introduce an additional factor of safety into the calculations.

* *Proceedings*, Am. Soc. C. E., September, 1923, p. 1619.

† *Loc. cit.*, August, 1923, p. 1317.

‡ *Loc. cit.*, May, 1923, pp. 871–882.

It is to be hoped that the work of the Department of Agriculture in collecting data on the position of the line of saturation in existing earth dams will include data on the effective size of the materials used in the dam and those found in the foundation, together with the porosities, depths to rock, and sub-surface conditions both up stream and down stream from the dam site. In the cases which have come to the attention of the writer, a large part of such data has been lacking. The mere locating of the line of saturation in an existing dam is not of much value in another design unless all the conditions are known.

Mr. Field* calls attention to the fact that a great deal of valuable data relative to failures and investigations of earth dams has been suppressed. It is only natural that companies which experience trouble with their dams should prefer not to have such accidents advertised, because the publicity is bound to affect their securities in an unfavorable manner. On the other hand, if all such accidents and failures could be freely discussed by these engineers who are most intimate with them, it would be of great advantage to the Engineering Profession. There would also be a material net advantage to the companies owning the structures.

Mr. Field considers that the writer has advocated the use of core-walls; others who have discussed the paper seem to feel that he has rather advised their omission. As a matter of fact, the writer has no hard and fast opinion about it. He believes that every case should be studied without prejudice and the decision, as to whether or not to use a core-wall, reached on the merits of each particular case. There are some cases where, in the writer's judgment, a core-wall most certainly should be used; there are other cases where economy might dictate either its use or its omission. In still other instances, the expense of a core-wall might be entirely wasted and the dam safer with the core-wall omitted.

Mr. Field does well in emphasizing the importance of drainage. Many dams otherwise unsafe can be made safe by proper provisions for drainage.

With respect to the hydraulic-fill and semi-hydraulic-fill methods, as questioned by Mr. Pratt,† the special problems of these types have been so fully covered in recent papers before the Society that any extensive discussion of their construction seemed to be superfluous.

Mr. Haydock‡ considers the writer to be rather severe in his criticism of the use of concrete paving as a cut-off. The method is quite general in distribution reservoirs of moderate height, especially where part of the slope is in excavation. Even in such case, however, the primary function of the slabs is to prevent the water from becoming muddy and to permit the ready cleaning of the reservoir. The writer has also lined several reservoirs in the manner described by Mr. Haydock, but his remarks on the subject had particular reference to higher structures, among which class there have been a number of instances where reliance on the water-tightness of such slope paving was later proved to be unwarranted by the facts. For this reason, the writer believes that it is safer to rely on such concrete paving only for protection against wave action.

* See p. 1874.

† *Proceedings*, Am. Soc. C. E., August, 1923, p. 1312.

‡ *Loc. cit.*, p. 1327.

TABLE 8.—FAILURES AND PARTIAL FAILURES OF EARTH DAMS.

Name of dam.	Location.	Height, in feet.	Date of failure.	Reference.	Core.	Remarks.
Lyman	Arizona	65	1915	*Eng. News*, v. 73, p. 794	Puddled	Piping and sloughing.
Calaveras*	California	240	1918	" " v. 72, p. 692	Clay	Internal liquid pressure.
Davis Reservoir	"	39	1914	" " v. 72, p. 106		No cut-offs on gate structure.
Débris Barrier, No. 1, Yuba River	"	..	1907	" " Aug. 8, 1907		Insufficient spillway.
Lake Francis	"	50	1899	*Trans.*, Am. Soc. C. E., Vol. LVIII (1907), p. 196	None	Settlement and seepage along outlet conduit.
Lower Otay	"	136	1916	*Eng. News*, v. 75, p. 334	Steel	Insufficient spillway.
Bonney	Delto, Colo	40	1908	*Eng. Record*, April 25, 1908		
Empire	Colorado	30	1909	State Engr.'s Rept	None	Settlement of outlet conduit.
Horse Creek	"	56	1914	*Eng. News*, v. 71, p. 828	"	Piping and sloughing.
Lake George	"	..	1914		Puddled	Piping.
Leroux Creek	"	25	1905	*Eng. News*, July 22, 1905		Insufficient spillway.
Lidderdale	"	19	1909	State Engr.'s Rept	None	" "
Standley Lake*	"	113	1914, '16	*Eng. News*, v. 78, p. 440	Puddled	Semi-hydraulic fill; sloughing.
Trout Lake	"	25	1909	State Engr.'s Rept	None	Insufficient spillway.
Victor	"	25	1901	*Eng. Record*, June 8, 1901		" "
West Julesburg	"	50	1910	State Engr.'s Rept	None	Seepage along ledge rock.
Ansonia	Connecticut	..	1894	*Eng. Record*, Nov. 10, 1894		Seepage along waste pipe.
Bridgeport	"	..	1905	" " July 22, 1905		Insufficient spillway.
Staffordville	"	26	1877		Masonry facing	Seepage along outlet conduit.
Wilmington	Delaware	..	1900	*Eng. Record*, Oct. 20, 1900	Brick facing	Seepage along pipe.
Portland	Maine	..	1893	*Eng. Record*, Aug. 19, 1893	Clay facing	Seepage along drain pipe.
Lynde Brook	Worcester, Mass	27	1876		Masonry facing	Seepage along outlet conduit.
Middlefield	Massachusetts	20	1901	*Eng. Record*, May 4, 1901	Rubble facing	Insufficient spillway.
Mill River	"	..	1874		None	Sloughing.
Mud Pond	East Lee, Mass	15	1886		"	Poor construction.
Grand Rapids	Michigan	30	1900	*Eng. Record*, July 14, 1900	Clay	Insufficient spillway.
Valentine	Nebraska	30	....		"	Concrete covered earth spillway.
Blue Water	New Mexico	35	1909	*Eng. News*, Sept. 30, 1909		Insufficient spillway.
Lake Avalon	"	48	1904	" " July 6, 1905		Piping.
Zuni	Black Rock, N. Mex.	70	1909	" " Dec. 2, 1909		Hydraulic-fill and rock; piping.
Melzingah No. 1	New York	30	1897	*Eng. Record*, July 17, 1897	Masonry	Insufficient spillway.
" No. 2	"	24	1897	" " " " "	"	" "
Norwich:						
Upper Dam	"	34	1905	*Eng. News*, June 29, 1905	Puddled	" "
Lower Dam	"	..	"	" " " " "		" "

TABLE 8.—(*Continued*).

Name of dam.	Location.	Height, in feet.	Date of failure.	Reference.	Core.	Remarks.
Schenectady	New York	80	1916	*Eng. News*, v. 76, p. 817	Timber	Insufficient spillway.
Six-Mile Creek	Ithaca, N. Y.	15	1905	" " June 29, 1905	....	" "
Lake Toxaway	North Carolina	62	1916	" " v. 76, p. 331	Masonry	....
Linville*	" "	160	1919	....	Clay	Internal liquid pressure.
Winston	" "	24	1912	*Eng. News*, v. 67, p. 667	Masonry	Insufficient spillway.
Lebanon	Ohio	30	1882	....	Masonry facing	" "
Tiffin	"	18	1913	....	None	Concrete covered earth spillway.
Portland	Oregon	..	1894	*Eng. Record*, Feb. 2, 1895	....	Concrete lining failed.
Conshohocken Hill	Pennsylvania	..	1873, '76, '79, '86	....	None	Clay lining gave away.
Jeanette	"	20	1903	*Eng. Record*, July 11, 1903	....	Insufficient spillway.
Johnstown	"	72	1889	*Eng. Record*, June 8, 15, 1889	None	" "
Lancaster	"	..	1894	" " Dec. 8, 1894	Puddled	Seepage along drain pipe.
Roxborough	Philadelphia, Pa.	..	"	" " Jan. 12, 1895	....	Settlement.
Scottdale	Pennsylvania	60	1904	*Eng. News*, Aug. 4, 1904	....	Piping.
Spartanburg	Oil City, Pa.	10	1892	....	None	Insufficient spillway.
Spring Lake	Fishkill, R. I	18	1889	*Eng. Record*, Aug. 31, 1889	"	Seepage.
Goose Creek	South Carolina	22	1916	*Eng. News*, v. 76, p. 232	"	Insufficient spillway.
Dallas	Texas	29	1891	....	Brick facing	Settlement.
Hatchtown	Utah	65	1914	....	....	Seepage along culvert.
Millville	"	36	1909	*Eng. Record*, v. 60, p. 324	Puddled	Piping through foundation.
Hatchtown	"	60	1914	....	Puddled	Ten to fifteen dams down stream destroyed.
Dells	Wisconsin	34	1911	*Eng. News*, v. 66, p. 483	Concrete	Insufficient spillway.
Hatfield	"	24	"	" " " "	"	" "
Hebron	Chico Rico Creek	56	"	....	....	Water through gopher holes.
Toronto	Canada	35	1912	*Eng. Record*, Apr. 27, 1912	Concrete	Insufficient spillway.
Bradford	Sheffield, England	90	1869	....	Puddled	Water followed outlet pipes.
Ashti	India	58	1883	Wegmann, p. 234	Puddled	Saturation of foundation.
Necaxa*	Mexico	193	1909	*Eng. Record*, v. 60, p. 1	Clay	Sloughing during construction.
Gatun*	Panama	115	1912	....	"	Internal liquid pressure.

* In the case of these dams, all of which were built by the semi-hydraulic-fill method, more or less extensive slides occurred during construction. All these dams were completed after the slides, and will be found in the list of successful dams (Table 7, *Proceedings*, Am. Soc. C. E., May, 1923, p. 913). The trouble with these dams could more properly be termed construction accidents rather than partial failures.

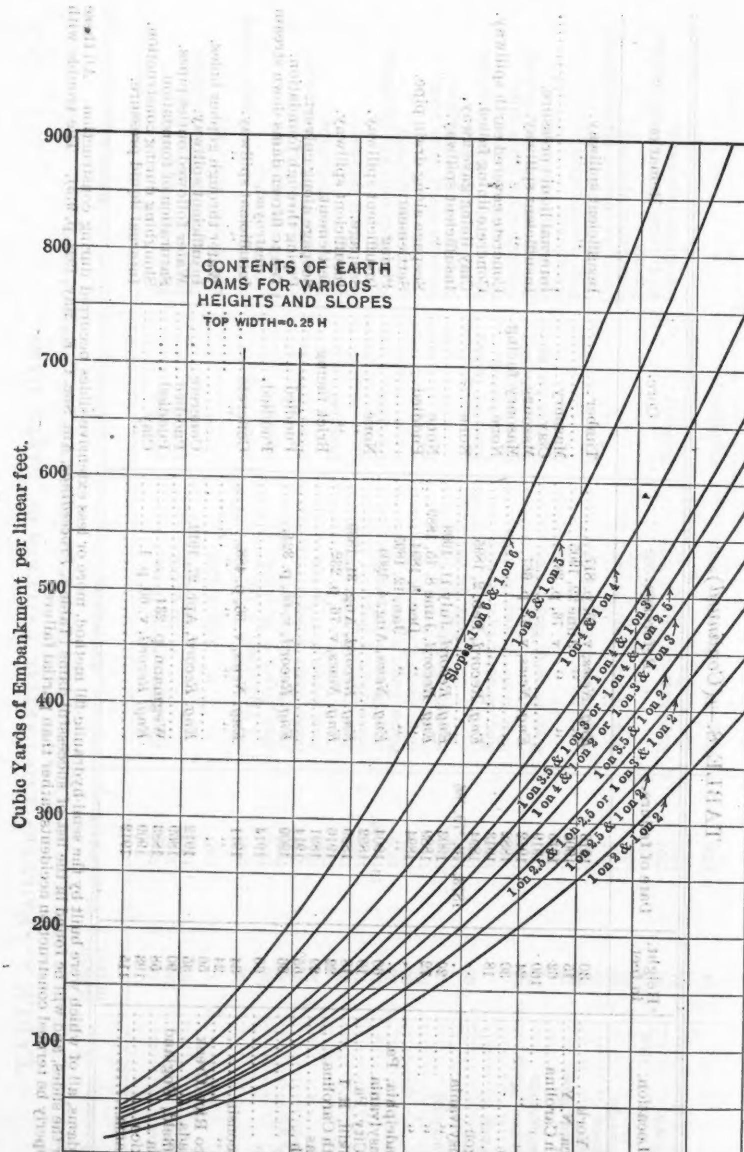
CONTENTS OF EARTH DAMS FOR VARIOUS HEIGHTS AND SLOPES
TOP WIDTH=0.25 H
Cubic Yards of Embankment per linear feet.
Height of Embankment in feet.
900
800
700
600
500
400
300
200
100
10
20
30
40
50
60
70
Slopes 1 on 6 & 1 on 6
1 on 5 & 1 on 5
1 on 4 & 1 on 4
1 on 4 & 1 on 3
1 on 4 & 1 on 2.5
1 on 3 & 1 on 3
1 on 3.5 & 1 on 3 or 1 on 4 & 1 on 2
1 on 3.5 & 1 on 2
1 on 3 & 1 on 2
1 on 2.5 & 1 on 2.5 or 1 on 3 & 1 on 2
1 on 2.5 & 1 on 2
1 on 2 & 1 on 2

FIG. 60.

Mr. Stevens* states that the writer has neglected to discuss one of the most important factors affecting the safety of earth dams, that is, the matter of foundations. Under Criterion 3, page 898,† the writer stated:

"A foundation soil that is stable and capable of sustaining a great load in its natural condition may become unstable when it becomes saturated after the dam is placed in service, and cause the sliding and subsidence of the slopes."

There follows a page relative to foundation conditions quite similar to those described by Mr. Stevens. The writer there emphasized the necessity for drainage under the conditions described by Mr. Stevens. Drainage was also discussed in detail on page 891.†

The data and figures which Mr. Stevens presents are valuable contributions to the discussion.

There is much to be gained by an examination into the causes of failures, and it is thought that Table 8 in which a number of failures of earth dams are listed, will prove to be of interest.

In making preliminary estimates on projects involving earth dams, the writer has found the formulas of Table 9 and the curves in Fig. 60 to be quite useful.

TABLE 9.—FORMULAS FOR VOLUME OF EARTH DAMS, IN CUBIC YARDS PER LINEAR FOOT.

(Top Width Equals 0.25 of the Height of the Dam.)

Slope of one face.	Slope of other face.	Volume.
1 on 2	1 on 2	$0.0833\ h^2$
1 on 2.5	1 on 2	$0.0926\ h^2$
1 on 2.5	1 on 2.5	$0.1021\ h^2$
1 on 3	1 on 2	$0.1020\ h^2$
1 on 3	1 on 3	$0.1203\ h^2$
1 on 3.5	1 on 2	$0.1110\ h^2$
1 on 3.5	1 on 3	$0.1296\ h^2$
1 on 4	1 on 2	$0.1208\ h^2$
1 on 4	1 on 2.5	$0.1295\ h^2$
1 on 4	1 on 3	$0.1387\ h^2$
1 on 4	1 on 4	$0.1574\ h^2$
1 on 5	1 on 5	$0.1944\ h^2$
1 on 6	1 on 6	$0.2313\ h^2$

h equals the height of the dam.

The provision for a sufficient spillway capacity for an earth dam is a most vital matter; probably more earth dams have failed from lack of such capacity than from any other one cause. The subject has not been treated to any extent by the writer because of the availability of the paper‡ by Weston E. Fuller, M. Am. Soc. C. E., which covered the subject very fully. Table 10 gives the maximum recorded floods on various American streams compiled from all available sources.

* See p. 1880.

† *Proceedings*, Am. Soc. C. E., May, 1923.

‡ "Flood Flows", *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.

TABLE 10.—Maximum Recorded Floods in American Rivers.

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second and per square mile.	No. of years observed.
Beacon Creek, near Fishkill, N. Y	0.25	3 200.00	18
Bulls Run, Long Level, Pa	0.58	4 170.00	....
Dooker's Hollow, North Braddock, Pa	0.6	4 000.00	....
Mann's Run, Creswell Station, Pa	0.67	2 540.00	....
Bedling Creek, near Utica, N. Y	1.13	120.40	Indefinite
Sylvan Glen Creek, New Hartford, N. Y	1.18	56.58	"
Mad Creek, LeRoy, N. Y	1.50	2 300.00	....
Cherry Vale Creek, Cherryvale, Kans	2.00	930.00	Indefinite
Indian Run, Letort, Pa	2.10	1 930.00	....
Canadochly Creek, East Prospect, Pa	2.2	1 630.00	....
Starch Factory Creek, near New Hartford, N. Y.	3.40	209.00	3
Estanguela, near Monterey, Mexico	3.50	825.00	Indefinite
Venison Branch, near Mulga, Ala	3.87	53.49	2
Riel's Creek, near Deerfield, N. Y	4.42	66.92	4
Mill Brook, Sherbourne, N. Y	5.00	262.00	....
Skinner Creek, Mannsville, N. Y	6.40	124.20	....
Cold Spring Brook, Mass	6.43	48.40	....
Blue Ribbon Creek, Pueblo, Colo	6.7	1 360.00	....
East Fork Honey Creek, New Carlisle, Pa	6.7	2 210.00	....
Camp Branch, Ensley, Ala	7.43	68.77	2
Osteen Arroyo, Pueblo, Colo	7.8	1 160.00	....
Croton, South Branch, N. Y	7.80	73.90	....
Woodhall Reservoir, Herkimer, N. Y	9.40	77.80	....
Mill Brook, Sherbourne, N. Y	9.40	241.00	Indefinite
East Fork, Honey Creek, New Carlisle, Pa	11.80	1 285.00	....
Goodyear Creek, Goodyear Bar, Calif	12.20	96.72	Indefinite
Stony Brook, Boston, Mass	12.70	121.00	"
Arroyo, Pueblo, Colo	15.8	619.00	....
Smartswood Lake, New Jersey	16.00	68.00	Indefinite
Alazon Creek, San Antonio, Tex	16.9	1 950.00	....
Martinez, San Antonio, Tex	19.6	1 223.00	....
Willow Creek, near Hepper, Ore	20.00	1 800.00	Indefinite
Croton, West Branch, N. Y	20.50	54.40	"
Beaver Dam Creek, Altman, N. Y	20.70	111.00	"
Cane Creek, Bakersville, N. C	22.00	1 341.00	"
Trout Brook, Centerville, N. Y	23.00	50.60	"
Apache Creek, San Antonio, Tex	24.00	947.00	....
Pinal Creek, Globe, Ariz	25.00	560.00	Indefinite
Pequonnoc, Bridgeport, Conn	25.0	157.00	"
Boggs Creek, Pueblo, Colo	26.5	582.00	"
Olmos Creek, San Antonio, Tex	26.8	930.00	"
Bear Grass Creek, Louisville, Ky	27.5	100.00	"
Wanuppa Lake, Fall River, Mass	28.5	72.00	"
Pinal Creek, Globe, Ariz	30.	440.00	"
Pequest, Huntsville, N. J	31.4	19.30	"
Peck Creek, Pueblo, Colo	34.4	564.00	"
Sawkill, near Merritt, N. J	35.0	228.60	"
Whippany River, Whippany, N. J	38.0	84.20	"
Cayadulta, Johnstown, N. Y	40.0	72.40	"
Basic Creek, Freehold, N. Y	41.0	81.22	"
Elkhorn Creek, Keystone, W. Va	44.0	1 363.00	"
Six-Mile Creek, Ithaca, N. Y	46.00	185.00	"
West Canada Creek, Motts Dam, N. Y	47.5	34.10	....
Sauquoit Creek, New York Mills, N. Y	51.5	53.40	....
Kosk Creek, near Henderson, Calif	51.9	44.32	2
North Fork, Cottonwood Creek, Oro, Calif	52.0	77.70	6
Rockaway, Dover, N. J	52.5	43.00	....
Onedia Creek, Kenwood, N. Y	59.0	41.20	....
Flat, Rhode Island	61.0	120.00	....
Pequanoc, Macopis, N. J	62.0	90.84	Indefinite
Camden Creek, Camden, N. Y	62.0	24.10	....
Nine-Mile Creek, Stittville, N. Y	62.6	124.90	....
Wissahickon Creek, Philadelphia, Pa	64.6	43.50	....
Sandy Creek, Allendale, N. Y	68.4	87.70	....
West Fork, Carson River, Woodfords, Calif	70.0	22.43	12
Loramie Reservoir Outlet, Ohio	72.0	97.22	Indefinite
Butte Creek, near Butte Valley, Calif	73.0	22.47	6
Rock Creek, Washington, D. C	77.5	126.30	....
Sudbury, Framington, Mass	78.00	41.88	....
Paquanock, Pompton, N. J	78.00	55.78	....
Putah Creek, near Guenoc, Calif	91.00	198.90	3
Independence Creek, Crandall, N. Y	93.20	66.50	....
Passaic, Chatham, N. J	100.00	17.20	....
Wanague, Pompton, N. J	101.00	83.61	Indefinite
Deer River, Deer River, N. Y	101.00	78.10	....
Wanaque, N. J	101.00	66.00	....

TABLE 10.—(*Continued*).

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second per square mile.	No. of years observed.
Tohickon Creek, Pleasant, Pa	102.00	112.50	....
Little Stony Creek, near Lodoga, Calif	102.00	69.22	5
East Branch, Fish Creek, Point Rock, N. Y	104.00	80.54	Indefinite
Onodago Creek, N. Y	108.00	30.00	"
Nashua, Mass	109.00	104.53	....
Sandy Creek, North Branch, Adams, N. Y	110.00	67.30	....
Lewistown Reservoir, Outlet, Ohio	111.00	57.65	Indefinite
Scantic, North Branch, Conn	118.00	51.80	....
Rockaway, Boonton, N. J	118.00	48.85	....
Ramapo, Mahwah, N. J	118.00	105.09	....
Laurel Hill Creek, Confluence, Pa	126.00	40.00	7
Santa Ysabel Creek, near Escondido, Calif	128.00	50.78	7
Patuxent, Laurel, Md	137.00	31.20	....
Nishaminy Creek, below Forks, Pa	139.00	97.60	....
Oriskany Creek, Colemans, N. Y	141.00	55.80	....
Devil's Creek, near Viele, Iowa	143.00	1 300.00	....
Oriskany Creek, Oriskany, N. Y	144.00	51.00	7
Turtle Creek, East Pittsburgh, Pa	146.00	64.21	....
Perkiomen Creek, Frederick, Md	152.00	69.20	....
Mohawk, Ridge Mills, N. Y	153.00	46.40	....
Rio Moca, below Moca, N. Mex	159.00	139.70	....
Ramapo, Pompton, N. J	160.00	65.88	....
Kinzua Creek, Dew Drop, Pa	162.00	19.94	2
East Branch, Fish Creek, Taberg, N. Y	169.00	65.09	16
Santa Ana, near Mentone, Calif	182.00	26.97	11
Fish Creek, West Branch, McConnellsville, N. Y	187.00	32.70	....
Tallulah, Tallulah Falls, Ga	191.00	40.63	4
Fishkill Creek, at Glenham, N. Y	198.00	69.19	3
Santa Ynez, near Santa Barbara, Calif	203.00	45.65	6
Unadilla, New Berlin, N. Y	204.00	40.00	....
Catskill Creek, South Cairo, N. Y	210.00	100.00	....
North Branch, French Creek Kemneytown, Pa	212.00	43.35	2
Arroyo Seco, near Soledad, Calif	215.00	61.86	12
Sapello, near Los Alamos, N. Mex	221.00	36.67	....
Salmon, Altmar, N. Y	221.00	27.60	....
San Gabriel, near Azusa, Calif	222.00	56.31	19
Alcovy, near Covington, Ga	228.00	9.52	4
Toccoa, near Blue Ridge, Ga	231.00	53.20	6
Esopus Creek, Olivebridge, N. Y	239.00	64.39	7
Cobbossecontic Stream, Gardiner, Me	240.00	13.65	17
Smith, Port Republic, Va	246.00	37.40	5
Susan, Susanville, Calif	256.00	7.03	6
East Canada Creek, Dolgeville, N. Y	256.00	54.30	16
Cohokea Creek, near Poag, Ill	259.00	13.90	3
Salmon, Pulaski, N. Y	260.00	41.65	16
Bear, Vantrent, Calif	263.00	98.10	9
Tule, near Portersville, Calif	266.00	20.41	12
Black, Forestport, N. Y	268.00	39.00	....
Millers Creek, near Lovella, Ore	270.00	24.93	9
Nottenly, Ranger, N. C	272.00	20.81	5
Crooked Creek, Heleman's Farm, Pa	279.00	43.37	2
Piscataquis, Foxcroft, Me	286.00	77.62	8
Brokenstraw Creek, Youngsville, Pa	290.00	24.50	2
Youghiogheny, Friendsville, Md	274.00	27.76	6
Antietem Creek, near Sharpsburg, Md	295.00	23.17	9
Oil Creek, Rouseville, Pa	302.00	27.71	2
San Luis Rey, near Pala, Calif	318.00	40.88	8
Silver Creek, near Lebanon, Ill	335.00	15.64	4
Croton, Croton Dam, N. Y	339.00	74.40	....
Carrabassett, North Anson, Me	340.00	40.21	5
Great, Westfield, Mass	350.00	151.90	....
East Canada Creek, Dolgeville, N. Y	356.00	24.70	....
West Canada Creek, Hinckley, N. Y	372.00	104.57	45
West Canada Creek, Trenton Falls, N. Y	376.00	96.54	16
Esopus Creek, Mt. Marion, N. Y	378.00	65.34	6
Rondout Creek, Rosendale, N. Y	380.00	51.34	12
Pompton, Two Bridges, N. J	380.00	61.60	....
East Fork, Carson, near Gardnerville, Nev	381.00	8.69	10
Black Lick Creek, Black Lick, Pa	386.00	50.82	8
Middle Oconee, near Athens, Ga	395.00	49.52	2
Calaveras, Jenny Lind, Calif	395.00	176.20	6
Pacolet, Spartanburg, S. C	400.00	88.90	....
Mohave, Victorville, Calif	400.00	33.58	7
Tygent's Valley, Belington, W. Va	403.00	40.88	5
Moose, Ayers Mill, N. Y	407.00	31.00	....

TABLE 10.—(*Continued*).

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second per square mile.	No. of years observed.
North Branch Potomac, Piedmont, W. Va	410.00	32.80	8
Hiwassee, Murphy, N. C	410.00	54.54	9
Mahoning Creek, Furnace Bridge, Pa	412.00	30.51	2
Rio Mora, Weber, N. Mex	422.00	65.70	....
South Fork, Sangamon, Taylorville, Iowa	427.00	9.70	4
Stony Creek, Johnstown, Pa	428.00	70.00	....
Youghiogheny, Confluence, Pa	435.00	52.07	7
Apalachee, near Buckhead, Ga	440.00	15.19	5
Whetstone, Bigston, S. D	441.00	2.95	9
Battenkill, Greenwich, N. Y	441.00	21.65	3
Casselman, Confluence, Pa	448.00	43.89	7
Teonesta Creek Nebraska, Pa	451.00	20.40	2
Cattarraugus Creek, Versailles, N. Y	467.00	53.53	....
Ausable, Ausable Forks N. Y	487.00	45.17	....
Cache Creek, Lower Lake, Calif	500.00	8.68	12
Deerfield, Shelbourne Falls, Mass	501.00	42.51	4
North Fork, Feather below Prattville, Calif	506.00	19.47	6
Olentangy, Columbus, Ga	514.00	70.00	Indefinite
West Canada Creek, Middleville, N. Y	518.00	24.90	....
Truckee, at Tahoe, Calif	519.00	2.60	14
Cosumnes, at Michigan Bar, Calif	524.00	42.75	6
Coosewatter, at Carters, Ga	531.00	31.92	10
Santa Caterina, at Monterrey, Mexico	544.00	590.00	Indefinite
Conewanger, Frewsburg, N. Y	550.00	20.95	2
Farmington, Conn	584.00	41.70	....
Tugaloo, near Madison, S. C	593.00	36.86	8
Stony Creek, near Fruto, Calif	601.00	48.75	12
Etowha, Canton, Ga	604.00	31.50	9
Hoosick, Johnsonville, N. Y	605.00	38.01	11
McCloud, near Gregory, Calif	608.00	68.26	8
Des Plaines Riverside, Ill	630.00	14.23	....
Mokelumne, near Clements, Calif	642.00	26.01	8
Manocacy, near Frederick, Md	660.00	31.00	11
Tuckasegu, Bryson, N. C	662.00	58.23	....
Little Tennessee, Judson, N. C	675.00	85.30	15
Raquette, at Piersfield, N. Y	723.00	8.13	4
Santa Ynez, near Lampac, Calif	725.00	28.14	5
Big Muddy, near Cambon, Ill	735.00	14.97	4
West Fork, Enterprise, W. Va	744.00	23.67	5
Broad, near Carlton, Ga	762.00	38.22	9
Passaic, Little Falls, N. J	773.00	24.20	....
Hudson, North Creek, N. Y	804.00	35.08	7
North, Point Republic, Va	804.00	29.69	5
Putah, Winters, Calif	805.00	37.27	8
Raritan, Bound Brook, N. J	806.00	64.52	96
Passaic, Dundee, N. J	823.00	43.38	95
Kettle, near Sandstone, Minn	825.00	7.15	4
North, Glasgow, Va	831.00	44.80	....
Minnesota, above Whetstone River, Minn	846.00	0.11	6
Dead, near The Forks, Me	870.00	20.74	5
Youghiogheny, below Confluence, Pa	874.00	52.63	32
Raritan, Bound Brook, N. J	879.00	59.30	....
Potomac, North Branch, Cumberland, Md	891.00	22.80	....
Flint, Molena, Ga	892.00	7.37	....
Black, Lyons Falls, N. Y	897.00	46.00	....
Schoharie Creek, Fort Hunter, N. Y	900.00	55.11	16
Clarion, Clarion, Pa	910.00	43.10	27
Stanislaus, Knights Ferry, Calif	935.00	61.18	10
Truckee, near Stateline, Calif	955.00	16.02	14
French Broad, Asheville, N. C	987.00	7.88	....
Flint, near Woodbury, Ga	988.00	30.62	6
North Fork, Shenandoah, near Riverton, Va	1 037.00	20.86	8
Scioto, Columbus, Ohio	1 047.00	80.82	Indefinite
Saluda, Watertown, S. C	1 056.00	12.08	....
Sacandaga, Hadley, N. Y	1 060.00	27.36	7
Genesee, Mount Morris, N. Y	1 070.00	39.20	....
French Creek, Carlton, Pa	1 070.00	22.98	4
Merced, near Merced Falls, Calif	1 090.00	34.18	11
East Branch, Penobscott, Grindstone, Me	1 100.00	23.86	8
Raquette, Massena Springs, N. Y	1 170.00	9.40	8
Neuse, Selma, N. C	1 175.00	6.70	....
Yuba, near Smartsville, Calif	1 220.00	90.91	10
Cache Creek, Yola, Calif	1 230.00	16.34	10
Mohawk, Little Falls, N. Y	1 306.00	26.65	16
Youghiogheny, Connellsville, Pa	1 320.00	27.50	4
Tygart Valley, Felterman, W. Va	1 327.00	26.36	5

TABLE 10.—(*Continued*).

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second per square mile.	No. of years observed.
Green Brier, Alderson, W. Va	1 344.00	41.60	....
Oconee, Corey, Ga	1 346.00	7.44	....
Cheat, Morgantown, W. Va	1 380.00	30.29	13
Genesee, Mt. Morris, N. Y	1 410.00	12.52	22
South Branch, Potomac, near Springfield, W. Va	1 440.00	17.81	8
Tuoloume, La Grange, Calif	1 500.00	35.07	18
Oostanaula, Resaca, Ga	1 527.00	14.50	....
Catawba, Catawba, N. C	1 535.00	61.89	10
Chattahoochie, Oakdale, Ga	1 560.00	31.28	10
South Fork, Shenandoah, near Front Royal, Va	1 570.00	48.92	8
Kennebec, Forks, Me	1 570.00	12.67	6
Oostanaula, Resaca, Ga	1 614.00	14.88	7
San Joaquin, Hamptonville, Calif	1 637.00	36.53	16
Allegheny, Red House, N. Y	1 640.00	25.00	8
Kiskiminetis, Avonmore, Pa	1 720.00	39.10	5
King's, near Sanger, Calif	1 740.00	25.25	18
Black, Carthage, N. Y	1 812.00	21.20	....
West Branch, Penobscot, Mellenochut, Me	1 880.00	12.90	9
American, Fair Oaks, Calif	1 910.00	55.00	9
Schuylkill, Fairmount, Pa	1 915.00	12.20	....
North Fork, Feather, Big Bend, Calif	1 940.00	56.34	6
Chemung, Elmira, N. Y	2 055.00	67.10	....
James Buchanan, Va	2 058.00	15.60	....
Tar, Tarboro, N. C	2 290.00	6.38	....
Androscoggin, Rumford Falls, Me	2 320.00	23.81	12
Kern, Bakersfield, Calif	2 345.00	4.15	19
Genesee, Rochester, N. Y	2 365.00	21.20	128
Menominee, near Iron Mountain, Mich	2 415.00	4.87	4
Ocmulgee, Macon, Ga	2 425.00	20.97	13
Chemung, Chemung, N. Y	2 440.00	21.52	11
Elkhorn, near Norfolk, Nebr	2 470.00	3.24	8
Sangamon, Riverton, Ill	2 500.00	7.50	4
Wisconsin, near Merrill, Wis	2 630.00	8.02	4
Kennebec, between Forks and Waterville, Me	2 700.00	48.56	14
Savannah, near Calhoun Falls, S. C	2 712.00	6.00	....
New, Radford, Va	2 725.00	63.78	13
Saline, Beverly, Kans	2 730.00	5.86	9
Hudson, Glens Falls, N. Y	2 760.00	25.36	15
Hudson, Fort Edward, N. Y	2 825.00	15.60	....
Catawba, near Rockhill, S. C	2 987.00	50.50	9
Shenandoah, Millville, W. Va	2 995.00	46.65	12
Verdigris, Liberty, Kans	3 067.00	16.45	10
Link, Klamath Falls, Ore	3 110.00	2.90	7
Klamath, Keno, Ore	3 150.00	2.68	7
Wabash, Logansport, Ind	3 163.00	17.99	21
Chattahoochee, West Point, Ga	3 300.00	26.86	10
Mohawk, Rexford, N. Y	3 384.00	23.10	....
Yadkin, Salisbury, N. C	3 399.00	40.00	....
Mohawk, Cohoes, N. Y	3 472.00	28.50	26
Staunton, Clarksville, Va	3 546.00	10.30	....
Crow Wing, near Mouth, Minn	3 580.00	2.85	3
Feather, Oroville, Calif	3 640.00	51.37	11
Neosho, Iola, Kans	3 670.00	20.33	9
Dan, Clarksville, Va	3 749.00	8.80	....
Tallapoosa, Milstead, Ala	3 840.00	9.50	....
Coosa, Rome, Ga	4 006.00	16.02	7
Pitt, Bieber, Calif	4 040.00	6.81	5
Merrimac, Lowell, Mass	4 085.00	19.80	....
Oconee, Dublin, Ga	4 182.00	8.35	8
Kennebec, Waterville, Me	4 270.00	35.36	14
Kennebec, Waterville, Me	4 410.00	25.20	....
Cape Fear, Fayetteville, N. C	4 493.00	16.30	....
Hudson, Mechanicsville, N. Y	4 500.00	26.67	26
Susquehanna, West Branch, Williamsport, Pa	4 500.00	11.60	....
Mississippi, above Sandy River, Minn	4 510.00	2.12	18
Merrimac, Lawrence, Mass	4 553.00	23.40	59
Broad, Alston, S. C	4 609.00	23.20	....
Yadkin, Norwood, N. C	4 614.00	13.70	....
Potomac, Dam No. 5, Md	4 640.00	22.20	....
Grand, Grand Rapids, Mich	4 900.00	8.04	....
Black Warrior, Tuscaloosa, Ala	4 900.00	38.80	....
Flint, Albany, Ga	5 000.00	7.79	4
Red Lake, Crookston, Minn	5 320.00	2.67	12
Monongahela, Lock No. 4, Pa	5 430.00	38.12	20
St. Croix, near St. Croix Falls, Minn	5 930.00	5.65	7

TABLE 10.—(*Continued*).

River.	Water-shed, in square miles.	Maximum discharge, in cubic feet per second per square mile.	No. of years observed.
Niobrara, near Valentine, Nebr.	6 070.00	1.15	7
Fox, Rapid Croche Dam, Wis.	6 200.00	2.49	10
New, Fayette, W. Va.	6 200.00	13.49	....
Rock, below Rockton, Ill.	6 290.00	4.31	7
Cedar, Cedar Rapids, Iowa	6 320.00	3.75	3
Delaware, Lambertsville, N. J.	6 500.00	53.80	120
Chippewa, Eau Claire, Wis.	6 740.00	9.00	4
Delaware, N. J.	6 750.00	50.00	....
Susquehanna, Northumberland, Pa.	6 800.00	17.50	....
Coosa, Riverside, Ala.	6 850.00	10.53	....
Mississippi	7 283.00	1.49	....
Savannah, Augusta, Ga.	7 500.00	40.00	66
Gunnison, Whitewater, Colo.	7 863.00	3.67	4
Smoky Hill, Ellsworth, Kans.	7 980.00	2.63	9
Grand, Palisade, Colo.	8 546.00	4.88	4
Connecticut, Holyoke, Mass.	8 660.00	21.10	....
Roanoke, at Neal, N. C.	8 717.00	7.38	....
Kanawha, Charleston, W. Va.	8 900.00	13.49	....
Allegheny, Kittanning, Pa.	9 010.00	26.66	8
Blue, near Manhattan, Kans.	9 490.00	7.25	9
Potomac, Point of Rocks, Md.	9 654.00	48.90	18
Susquehanna, Wilkes-Barre, Pa.	9 810.00	22.20	7
Connecticut, Hartford, Conn.	10 234.00	20.30	105
Sacramento, Red Bluff, Calif.	10 410.00	24.42	11
Potomac, Md.	11 043.00	42.60	....
Potomac, Great Falls, Md.	11 427.00	41.20	....
Loup, Columbus, Nebr.	13 540.00	5.17	12
Minnesota, near Mankato, Minn.	14 600.00	3.0	10
Alabama, Selma, Ala.	15 400.00	9.5	14
Illinois, Peoria, Ill.	15 700.00	3.66	16
Mississippi, Anoka, Minn.	17 100.00	2.87	9
Ohio, Pittsburgh, Pa.	19 100.00	22.98	30
Tennessee, Chattanooga, Tenn.	21 382.00	34.37	38
Republican, Bostwick, Nebr.	22 300.00	1.10	11
Ohio, Wheeling, W. Va.	23 800.00	20.80	22
Susquehanna, Harrisburg, Pa.	24 030.00	30.60	41
North Platte, Camp Clarke, Nebr.	24 800.00	0.95	11
Red, Grand Forks, N. D.	25 000.00	1.70	31
Mississippi, St. Paul, Minn.	36 085.00	19.73	46
Colorado, Austin, Tex.	37 000.00	3.33	9
Mississippi, Prescott, Wis.	44 070.00	2.50	6
Platte, near Columbus, Nebr.	56 900.00	0.83	12
Kansas, Lawrence, Kans.	59 841.00	3.80	25
Mississippi, Clayton, Iowa	79 040.00	2.66	6
Red, Arkansas	97 000.00	2.32	....
Mississippi, Grafton, Ill.	171 570.00	2.10	6
Ohio, Paducah, Ky.	205 750.00	7.00	6
Colorado, Yuma, Ariz.	225 000.00	0.67	11
Missouri, Sioux City, Iowa	323 462.00	1.64	6
Missouri, St. Charles, Mo.	530 810.00	1.13	6
Mississippi, St. Louis, Mo.	702 380.00	1.28	6

PROPOSED LOADING FOR HIGHWAY BRIDGES

Discussion*

BY MESSRS. J. A. L. WADDELL, OTIS E. HOVEY, D. B. STEINMAN, R. DE CHARMS, SAMUEL J. OTT, LEWIS E. MOORE, AND GLENN B. WOODRUFF.

J. A. L. WADDELL,† M. AM. SOC. C. E.—In the speaker's opinion, this paper will be of considerable value in bringing to the attention of members the investigations on the question of live loads for highway bridges being made by other Societies, and will give those interested an opportunity to discuss such work.

In contrasting the author's proposed live loading with that of the Tentative General Specifications for Steel Highway Bridges of the American Railway Engineering Association,‡ the speaker's opinion may be somewhat prejudiced, on account of the fact that he and his Associate Engineer, Shortridge Hardesty, Assoc. M. Am. Soc. C. E., have had considerable to do with the preparation of the latter.

In December, 1922, the American Association of State Highway Officials issued some "Standard Specifications for Steel Highway Bridges". The live loading of these specifications seems to have been accepted by the Special Committee on Specifications for Bridge Design and Construction of the Society and published.§

The speaker prefers the A. R. E. A. specifications to those of the author, his reasons being as follows:

A.—Although no ordinary truck load exceeds 9 ft. in width, 10 ft. should be allowed as the minimum distance between adjacent center lines of traffic, so as to permit passing at reasonably high speed. If an attempt is made to drive any closer, the drivers of both vehicles will automatically lower their speeds, which procedure, from the point of view of safety, is as it should be. With the reduction of speed, however, there is a drop in the percentage of impact; hence, two lines of vehicles occupying a total width of 18 ft. and running at slow speed would probably produce no greater live loading effect than the same two lines of vehicles occupying a total width of 20 ft. and running at a materially higher speed.

Many modern highway bridges have clear roadways of 18 ft., and necessarily must accommodate double lines of traffic; but for future constructions it is advisable to allow 10 ft. for each line of vehicles. In general, it may

* This discussion (of the paper by Harold D. Hussey, Esq., published in the August, 1923, *Proceedings*, and presented at the meeting of September 5, 1923), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York City.

‡ *Bulletin No. 252*, Am. Ry. Eng. Assoc., Vol. 24 (December, 1922), pp. 162–164.

§ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1379.

be stated that widths of highway bridges, measured between faces of wheel guards, should be multiples of 10 ft.; for instance, there is no special advantage in a 25-ft. clear roadway over one of 20 ft. This claim gradually loses its importance as the clear width increases, for the reason that many narrow vehicles are in common use. For instance, a 45-ft. clear roadway might often permit five lines of vehicles on the structure, even if they were too closely packed for rapid driving; hence, in this case, the extra 5 ft. would not be wasted.

B.—Each of the author's suggested standard live loads has a single motor truck at its head, as compared with the A. R. E. A. load of three trucks. It is possible that three may be really more than necessary, but certainly there should be at least two. The speaker is of the opinion that on certain bridges in large cities a line of three consecutive heavy trucks is not unusual, and that the bridge loads in such cities are gradually increasing.

C.—The author spaces his motor trucks longitudinally 50 ft. on centers, while the A. R. E. A. specifications make the spacing 33 ft. With the longer spacing, greater speed can be used, as was the case with the wider roadway. The speaker is of the opinion that it is better to adopt the closer spacing.

D.—Both the author and the A. R. E. A. specify three classes of live loads. When these loads are properly compared there is really not much difference between the corresponding effects produced. Mr. Hussey's three classes are direct decimal multiples of each other, which sometimes affords an advantage in calculating; but those of the A. R. E. A., in the speaker's opinion, are more logically graded to meet the actual conditions of the several kinds of traffic.

E.—A point of superiority of the A. R. E. A. specifications is the reduction of live load per square foot of floor for increased width of roadway. This is in accordance with the "probabilities" and results in true economy, because the greater the number of vehicles on a fully covered roadway, the lighter will probably be the average weight of a vehicle, and, therefore, the smaller the live load per square foot of floor.

There is one good feature of the A. R. E. A. loadings to which attention should be called, namely, the alternative loading of a single heavy truck for certain classes, that truck being the same as the standard truck of the preceding heavier class. It is not contemplated to allow passing two such heavy trucks on the bridge, but all parts are to be designed to carry the one heavy load placed in the critical position for the part considered. This will affect principally the stringers.

Comparing the A. R. E. A. specifications for live loading with those of the American Association of State Highway Officials, it is evident that those of the A. R. E. A. are worked out more systematically, consist of a smaller number of standards, are easier of application in computation, and require lighter electric railway loadings than those of the Highway Association. The latter specifications give two classes of electric cars, the first exceeding that of the A. R. E. A. in the ratio of 11 : 10 and the second in the ratio of 22 : 10. The second loading is that of ordinary freight cars, while the first

loading and that of the A. R. E. A. are of urban passenger cars. In general, there is no great difference in the highway live loadings for these two specifications.

There is one decidedly objectionable feature in the A. A. S. H. O. loadings, namely, the surrounding of a truck load by a uniform load. This requirement was certainly not inserted by any bridge designer, for the complications involved in its application are simply infernal.

The A. R. E. A. specifications are limited to spans of less than 300 ft., but there appears to be no such limit in the A. A. S. H. O. specifications. The placing of such a limit is a wise move, because any span of 300 ft. or more is of such importance as to warrant calling in a bridge specialist to do the designing. In that case, the determination of the amounts of the various loadings should be left to his judgment.

Whatever system of live loads is adopted eventually, it will be necessary for designers to have at least two curves of equivalent uniform loads, one set for shears and another for moments, because any attempt to utilize a single average set for both shears and moments will give variations that are too great to be ignored. Dr. Steinman's excellent method of adjustment of live load to length of span covered by moving load, evolved for railway bridges,* could then be applied advantageously to highway bridges also.

The speaker is convinced of the necessity for co-operation between the Bridge Committees of the A. R. E. A., the A. A. S. H. O., and the Society, in order that they may come to an agreement on a single specification for highway bridges. Due regard should be given to the voice of the computing engineers, who should be represented, for on them will fall the burden of applying any unnecessary refinements.

The speaker remembers the time when it was customary for chief engineers of railroads to designate as many as five engine loadings, making competitors calculate each member for each of the live loads. In fixing highway bridge specifications, all such hair-splitting must be avoided, as it is useless and uneconomic.

OTIS E. HOVEY,† M. AM. SOC. C. E.—The speaker is familiar with the investigation of highway bridge loads made by Mr. Hussey, which was the basis of his paper. The study was undertaken and carried out with thoroughness, and was an earnest endeavor to arrive at a suitable and convenient system of highway bridge loads.

For several years, the speaker has believed that material improvement in the specifications for highway loads was feasible and desirable. The use of concentrated wagon, truck, or road-roller loads for stringers, floor-beams, and similar members, combined with uniform loads per square foot around and following them, is cumbersome and inconvenient. In the Tentative Specifications for Steel Highway Bridge Superstructure,‡ proposed by the Special Committee on Specifications for Bridge Design and Construction of the

* *Proceedings*, Am. Soc. C. E., May, 1922, p. 1043.
† Asst. Chf. Engr., Am. Bridge Co., New York, N. Y.
‡ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1377.

Society, the uniform loads for girders and trusses more than 50 ft. long are varied with the class of structure and with the loaded length according to arbitrary rules determined from a study of the equivalent uniform loads corresponding to certain trucks in various combinations. Each loaded length for each class of bridge has a specific uniform load per square foot. As a member of the Committee which prepared the Tentative Specifications, the speaker shares the responsibility for the system proposed, but he feels that a much better and more convenient system can readily be developed.

The speaker would like to see highway loads as thoroughly systematized as railroad loads were, after the late Theodore Cooper, M. Am. Soc. C. E., proposed his E-loadings in 1894. Previous to that time, railroad loads had been in a state of development that had led to confusion.

The Specifications for Design and Construction of Steel Railway Bridge Superstructure* as submitted by the Special Committee on Specifications for Bridge Design and Construction, of the Society, continues the use of the Cooper E-system of loads, but also provides for the more modern M-loadings, proposed by D. B. Steinman, M. Am. Soc. C. E.† Such systems of loads are convenient in use, well represent actual locomotive and train loads, and afford standards of comparison that are of much value when rating old structures.

The speaker believes that any rational system of highway bridge loads should be based on a load in a lane of traffic. Many tests and observations show that this lane should be at least 9 ft. wide, in order that traffic may move freely in the same, or opposite, directions in the various lanes comprising the clear width of roadway. The author proposes that his loads be based on 9-ft. lanes of traffic, and the speaker agrees with his conclusion. The roadway widths should then be in multiples of 9 ft., in order to give the greatest economy and service. When the bridge has a clear roadway which is not a multiple of 9 ft., it seems satisfactory to use loads on the trusses proportioned to the actual clear roadway width, as proposed by the author.

Admitting that a load in a lane of traffic is the most desirable basis, it remains to inquire whether Mr. Hussey has selected the best and most expedient form of load. His investigation, based on track loads, resulted in a system that agrees with average traffic conditions more closely than the complicated one proposed by the Special Committee of the Society. His loads for different classes of bridges are proportional to a basic load, T-10, on a traffic lane 9 ft. wide, for which stress calculations become very simple, provided that a set of moment and shear tables is prepared for the basic load, as has been done in the railway specifications for both E-10 and M-10. The calculations would then reduce practically to the proper selection of moments and shears from the tables, and the application of the proper constant factors for any class of bridge and clear width of roadway. Had this, or a similar loading, been adopted by the Special Committee on Specifications for Bridge Design and Construction, the tables would have been prepared and printed.

It is the speaker's opinion that the proposed loads and methods of applying them constitute a distinct advance beyond present specifications and practice.

* *Proceedings*, Am. Soc. C. E., January, 1923, Papers and Discussions, p. 53.
† *Loc. cit.*, May, 1922, p. 1043.

D. B. STEINMAN,* M. AM. SOC. C. E.—This paper is a valuable contribution to bridge specifications, representing another step in the direction of standardization and simplification. Even if modified in details, the suggestions embodied in the paper should help to place the matter of highway loading specifications on a more definite and scientific basis. After all the variation and ambiguity in specifying highway loadings in the past, this new proposal should be welcome.

Although the new specification for truck loading proposed by the American Railway Engineering Association† has a number of points of resemblance to Mr. Hussey's proposed specification, the latter possesses several important advantages as follows:

1.—It offers a simpler loading diagram, with two axle concentrations instead of six.

2.—It affords the advantage of proportionality between the classes of different weight, permitting the change from one class to another by the application of a simple multiplier for all stresses.

3.—It offers a convenient and scientific method of providing for all widths of roadway.

In comparison with the new specification for highway loading proposed by the Special Committee on Specifications for Bridge Design and Construction of the Society,‡ the author's proposal possesses the obvious advantages of simplicity, ease of application, and generality of scope. The Tentative Specification submitted by the Committee does not provide for roadways of unusual width, such as occur on larger bridges.

In regard to the details of the loading diagram proposed by Mr. Hussey, the speaker wishes to offer a few suggestions. Referring to Fig. 2,§ it will be noted that the author's loading diagram consists of a motor truck followed by a uniform load of 720 lb. per lin. ft. for Class A loading, 630 lb. for Class B and 540 lb. for Class D. The speaker would suggest that these values might be rounded off advantageously, changing them to 800, 700, and 600 lb. per lin. ft., respectively. In connection with this change, the roadway width per line of traffic might be increased advantageously from 9 ft. to 10 ft. This would preserve unchanged the intensity of the uniform load, and would mean only a small and permissible reduction in the effect of the advance concentrations. Besides the advantages of convenience from rounding off the figures in the loading diagram, the adoption of a basic width of 10 ft. (instead of 9 ft.) per line of traffic, may, by suggestion, be conducive to more generous allowances of width for bridge roadways. The use of round figures for a highway loading specification is desirable for simplicity; greater precision is not warranted under the circumstances.

The three loading classes proposed by the author‖ are T-20 for Class A, T-17.5 for Class B, and T-15 for Class C. The speaker believes that a wider

* Cons. Engr., New York, N. Y.

† *Bulletin No. 252*, Am. Ry. Eng. Assoc., p. 163.

‡ *Proceedings*, September, 1923, pp. 1380–1382.

§ *Loc. cit.*, August, 1923, p. 1033.

‖ *Loc. cit.*, p. 1034.

range is desirable to cover the diversity of conditions encountered in bridge designing, and, therefore, would suggest three substitute classes, of T-20, T-15, and T-10, respectively. In lieu of this change, a fourth class (Class D) might be added to the author's list, which class would be T-10 loading. This class would provide for light bridges in undeveloped country, including foreign bridges. For export work, it frequently is necessary to design bridges for maximum loads represented by 6-ton wagons, so that a T-10 loading specification would provide amply for future traffic development.

The author's proposal to take care of varying roadway widths by uniformly varying coefficients, as indicated in Fig. 4,* appears to be an excellent idea. It provides a uniform increase of stresses with roadway widths, instead of breaks or gaps at intermediate widths. Besides simplifying the calculations for various widths, it possesses the advantage of eliminating a temptation to provide insufficient roadway width. (If a specification corresponding to the broken line in Fig. 4 were adopted, it might offer an inducement to stint the width of roadways, because of the considerable jump in cost between such widths as 17 and 18 ft., or 25 and 27 ft.) With the use of a straight line specification for width coefficients like Fig. 4, there is no obstacle to the change of the basic width from 9 ft. to 10 ft.; in fact, there would be an advantage inasmuch as the coefficient for any width of roadway would be simply given as one-tenth of the width.

Another suggestion offered by the speaker is that special consideration be given to long-span bridges, say, to spans of more than 300 ft. For these spans, it would be justifiable to drop the advance concentrations of the proposed T-loading, and to use only the following (uniform) load. Designers will almost certainly ignore the advance concentrations in calculating long spans, and engineers may as well face that fact by specifying that the simple uniform load may be used for spans of more than 300 ft.

With an increase of span length and of roadway width, the probability of maximum loading over the entire area of the roadway obviously diminishes. The author's loading diagram provides, in a measure, for the reduction of probable load with increasing span, at least for the usual range of span lengths. Some provision ought to be made, however, for the reduction of the probable load with increasing roadway width. On wide bridges, with four or more lines of traffic, there is certainly less probability of full loading than on bridges providing for only one or two lines of traffic. For roadway widths of 60 to 150 ft., as in some of the long-span designs, it appears unreasonable to assume the same intensity of loading as on ordinary, narrow bridges. Allowance for this condition might be made by changing the straight-line graph of width coefficients (Fig. 4) to a line of flatter slope or, better still, to a parabolic curve. As a tentative practical proposal, the speaker would suggest retaining the author's straight line for widths up to 40 ft., and then breaking it to continue as a straight line of half the slope. This may be represented by a formula, $K = \frac{W}{10}$, for widths up to 40 ft., and $K = 2 + \frac{W}{20}$,

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1036.

for widths of more than 40 ft., in which K is the coefficient for truss loads and W is the width of roadway, in feet, corresponding to the co-ordinates of the author's Fig. 4.

The problem of constructing a standardized highway loading specification cannot be solved as definitely as the corresponding problem for railway loadings, because, in the case of highway loading, the probability factor enters more largely. The train loading on a railway bridge is comparatively definite. In evaluating a highway loading, however, the number of lines of traffic, the spacing of trucks, and the proportion and distribution of heavy trucks, are all variables which add to the uncertainty of the problem, reducing the probability of a maximum, and leaving more to engineering judgment in fixing the loading. These considerations should be kept in mind in studying the suggestions for modification which the speaker has ventured to offer. At the same time, they indicate the difficulties of the problem which Mr. Hussey has attacked, and the importance and value of his paper as the first consistent and practical solution of this problem.

R. de Charms,* Assoc. M. Am. Soc. C. E.—The subject of loading for highway bridges is an important one to the profession at present, because the growth of motor transportation has been so sudden that engineers, confronted with the problem of designing adequate bridges for loadings different from the road roller, have had to rely on their own resources, and the result has been a multiplicity of specifications and loadings with little attempt at standardization. In his valuable paper, Mr. Hussey has studied the problem and has blazed the way for standardizing the live load for highway bridges as Cooper did for railroad bridges twenty-eight years ago.

Highway bridge loadings, however, differ from railroad bridge loadings. A railroad bridge is built for a definite engine, and the railroad management can control the traffic so as not to overstress the bridge. On the other hand, although a highway bridge is built for a certain truck load, and the capacity of the bridge is posted, it generally happens that no attention is paid to the sign, and that when an accident occurs the engineer is blamed for not designing the bridge for the traffic. Of the two, the highway bridge, therefore, has to have a greater factor of safety and this should be kept in mind in preparing a specification that will stand the test of time.

The author has taken care of future development in his system of loadings by providing for tables using a basic load that can be increased by simple multiplication. He has, however, limited his system by specifying a 9-ft. width for one line of traffic. The changes in sizes and weights of trucks in the last year or two indicate that a 9-ft. width for clearance may soon be inadequate. The width is not limited by a definite gauge line as in railroad work; with the increases in truck sizes, the width and length are more apt to be changed than the height. It is true that the system proposed by the author can be adjusted to various widths of roadway and that if the load is increased proportionally to the width there will be no difficulty in adjusting it to the larger sized trucks of the future.

* Asst. Bridge Engr., Bridge Div., State Highway Comm. of New Jersey, Newark, N. J.

The proposed loadings call for a T-20 truck for Class A bridges carrying city and primary highway traffic. In studying the paper, the speaker wondered if this is heavy enough to take care of the development of traffic during the next ten or fifteen years. Several present-day specifications give loadings heavier than T-20, including the proposed American Railway Engineering Association's "Tentative Specifications for Steel Highway Bridges", which give a loading of three 20-ton trucks with similar wheel-spacing and concentrations.

As a matter of interest pertinent to this discussion, the equivalent uniform load curves for maximum shear and for moment at the quarter-point for the A. R. E. A., 20-ton loading have been plotted. (Fig. 5.) These curves show a maximum divergence of about 30 lb. between equivalent uniform load for shear and moment. In this particular, the loading does not give as economical a design as that of the author, which shows a maximum divergence of about 15 lb. However, the curves do show that the equivalent uniform load for the A. R. E. A. is considerably heavier than the proposed T-20 loading.

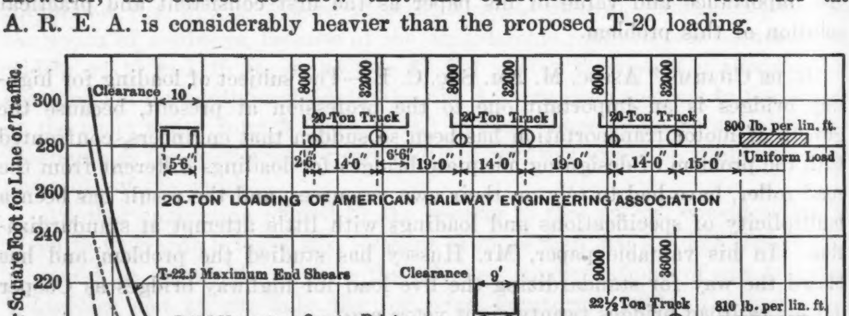

Fig. 5.

For further comparison two curves have been plotted from Mr. Hussey's system to conform as nearly as possible to the A. R. E. A. specification curves. (Fig. 5.) These curves show that the equivalent uniform load values for T-22.5 are similar to those derived from the A. R. E. A. specification for 20-ton truck loading. For maximum end shear the T-22.5 loading gives higher values for spans of less than 100 ft. and is practically the same for spans from 100 to 300 ft. Of the two types shown on Fig. 5 the T-22.5 loading is the more regular one and shows closer agreement between the equivalent uniform load for shear and for moment. These curves have been plotted to show that Mr.

Hussey's system is adaptable to heavier loadings and that it is applicable, therefore, for the future development of traffic; it is hoped they will add to the valuable information contained in the paper.

SAMUEL J. OTT,* M. AM. SOC. C. E.—This paper is a brief summary of a large amount of work in reviewing the live load assumptions used in the past for designing highway bridges, in studying the effects of the loadings and attempting to reconcile their inconsistencies, and in endeavoring to evolve a substitute system that would be simple in application, consistent as to effect, and, above all, one that would square with the character of present-day traffic.

The proposed loading system should have a very strong appeal to engineers engaged in actual bridge computations, because it is so workable. The similarity of this system to those long in use on railroad work should strongly commend it, especially to those engineers whose experience has covered the fields of railroad as well as of highway bridges.

The basic idea in the development of this loading is that present-day motor vehicles move in lanes of traffic; the analogy between this loading and the conventional railroad loadings is immediately apparent.

In the past, an assumption for designing purposes of a loading, in pounds per square foot of floor area, was probably as fair as any other; but, to-day, if engineers are willing to recognize the fact that highway traffic has undergone a complete change during the past decade, they must revise their conceptions accordingly.

From the standpoint of the computer, the various systems of loading now specified have been more or less open to question as to meaning, and, generally, they have been cumbersome in application, as compared with the conventional railroad loadings. Starting, then, from the idea of lanes of traffic, there is no reason why a highway loading system should not be developed along the same lines as railroad loadings. The simplicity of the proposed loading is appreciated when it is considered that the system can be expressed by a simple diagram, whereas the older ones required two or three pages. By contrast, in this connection, the speaker respectfully refers the reader to the Tentative Specifications for Steel Highway Bridge Superstructure,† submitted as a Progress Report by the Special Committee on Specifications for Bridge Design and Construction.

The most attractive feature, however, of the new system is that it is rational, and that its tendency is to produce more consistent results in designing. The older type, with its requirements for various kinds of loading for the different members of the same structure, must admittedly be only an arbitrary approximation when it is considered that all the members of that structure are subject to the effects of the same actual loads.

This loading has so many advantages, that it is hoped that when the Society shall finally adopt a specification for highway bridges, it will prescribe this system as the basis for design, rather than the cumbersome one set forth in the Tentative Specifications referred to previously.

* Rutherford, N. J.

† *Proceedings*, Am. Soc. C. E., September, 1923, p. 1377.

LEWIS E. MOORE,* ASSOC. M. AM. SOC. C. E. (by letter).†—A discussion of the question of highway bridge loadings should begin with the consideration of how large actual average highway loads are at the present time.

The writer has given considerable attention to this particular phase of the subject and, as a result, has reached the conclusion that on an average the loads from passenger automobiles in the densest traffic do not amount to more than 30 or 35 lb. per sq. ft. of roadway; for example, the writer's own touring car weighs 3 290 lb. ready for the road, without passengers, and about 4 000 lb. with 5 passengers. It is 5 ft. 6 in. wide and 15 ft. long over bumpers, and thus covers an area of a little more than 80 sq. ft., which is equivalent to 50 lb. per sq. ft. Another touring car of a different make weighs 3 800 lb. without passengers, and about 5 000 lb. when fully loaded. It is 18 ft. over bumpers and thus covers a roadway area of 100 sq. ft., which, again, gives an average weight of 50 lb. per sq. ft.

If it were possible to pack a roadway with automobiles touching each other on all sides, the result would be an average load of 50 lb. per sq. ft., with occasional heavier concentrations, due to trucks. As a matter of fact, cars are never driven touching each other, either end to end, or side by side. For instance, a 40-ft. roadway will accommodate only four lines of traffic. Looking at this in another way, the two cars previously noted weigh 266 and 278 lb. per lin. ft., respectively. Four lines of this loading will weigh 1 080 lb. per lin. ft., which, divided by the width of a 40-ft. roadway, gives an average of 27 lb. per sq. ft., assuming that each car is touching the next preceding one.

A 25-ft. roadway will accommodate three lines of traffic by crowding, which will weigh about 32 lb. per sq. ft. Some cars are heavier than those mentioned, but this added weight is offset by the fact that, as driven, cars have a considerable distance between each other, front and rear.

The previous discussion indicates that the present type of passenger cars will give an average load distribution of about 30 lb. per sq. ft. on the roadway. From this, it appears that in designing new bridges the author's assumed loading of 80 lb. per sq. ft. is sufficient. If applied over a 9-ft. width, it amounts to 720 lb. per lin. ft., or two and two-thirds times the weight per linear foot of the passenger cars described. This would appear to be an ample margin of safety for passenger traffic. The use of trucks introduces another condition, as pointed out by the author. The large margin in the assumed load of 80 lb. per sq. ft. makes it apparent that a bridge designed for this load, all parts of which are capable of carrying the concentrations produced by one, or at most two, 20-ton trucks, would be sufficient, as far as one can tell, for all probable future highway demands.

It is quite likely that developments in hauling will tend toward the use of trailers rather than toward increased weights of the trucks themselves, for two reasons: First, the restriction on the amount of single concentrated loads allowed on highways; and, second, the greater flexibility of trailer operation, speaking only from the standpoint of freight carrying.

* Cons. Engr., Boston, Mass.

† Received by the Secretary, September 13, 1923.

The revision of former highway bridge loadings is quite desirable, but there is some question as to whether the author's proposed loading meets the issue. His loading very much resembles in principle the railroad loading of a Cooper engine, followed by a uniform load. As a matter of fact, on highway bridges, the uniform load is likely to be both before and behind the truck, which is a different condition from that assumed by the author. For the floor, the problem is very simple, as the truck concentrations would cover its design. For the trusses, the problem is somewhat different, and the writer ventures to suggest that it would be not only simpler, but also as near the actual conditions, to use a uniform load of 80 lb. per sq. ft. over 9 or 10 ft. of width, plus a single concentrated load, which can be placed anywhere. This is the old locomotive excess method in railroad work, which was discarded long since in favor of the Cooper loading. It lends itself, however, peculiarly well to the problem in question, because the excess load can be put anywhere and actually does occur anywhere in highway bridges.

To show how this would apply, Table 2 has been prepared. The panel concentrations for the loadings given in the paper have been computed for 10, 12, 15, and 18-ft. panels, including the proper part of the uniform load and of the load of the other axle. These concentrations are given in Column (2). Panel concentrations have also been computed for the uniform load only and appear in Column (3). The difference between the two concentrations for each panel length gives the excess of truck load over uniform load, as shown in Column (4). The part of Table 2 for the 8 000-lb. load, is similar; showing that the excess is only 800 lb. for a 10-ft. span and becomes negative for a 12-ft. span. It may be disregarded.

TABLE 2.—PANEL CONCENTRATIONS, TRUCK, UNIFORM, AND EXCESS.
(Single Line of T-20 Loading.)

Panel length, in feet.	PANEL CONCENTRATIONS, IN POUNDS.		
	Truck.	Uniform live loads.	Excess.
(1)	(2)	(3)	(4)
	32 000-LB. AXLE.		
10	32 000	7 200	24 800
12	32 120	8 640	23 480
15	32 600	10 800	21 800
18	34 620	12 960	21 660
	8 000-LB. AXLE.		
10	8 000	7 200	800
12	8 000	8 640	Negative.

A study of Table 2 shows that a uniform load of 720 lb. per lin. ft., with an excess varying from 24 800 to 21 660 lb., would provide for most of the cases occurring in practice.

The writer believes that the use of any such variable excess is not justified, in view of the uncertainties as to the actual loads which come on the

bridge, and that the use of a single excess of 24 000 or 25 000 lb., in addition to the 720 lb. per lin. ft., would approach practical conditions as closely as is reasonably warranted. In other words, in designing trusses where loads are so variable and uncertain, there seems to be little advantage in going to the great refinement of using certain particular concentrated loads, spaced a certain distance apart and followed by a uniform load. Actual conditions, of course, vary greatly from the assumed conditions and, although the author's loads are intended to, and probably do, represent a maximum, they also represent an unnecessary refinement for truss design.

In truss design, it would seem to the writer sufficient to use for each line of traffic a uniform load of 720 (or 800) lb. per lin. ft., with a single concentrated excess of 25 000 lb. placed anywhere. The resulting bridge would be strong enough in all its details to carry the 20-ton truck and also strong enough in all its main members to carry such trucks at any reasonable distance apart.

In passing, it should be noted that if the 80 lb. per sq. ft. is taken over a 10-ft. width of roadway, instead of the 9-ft. width used by the author, it will amount to just 800 lb. per lin. ft. of bridge. If the truck assumed weighs 40 000 lb. and if that be divided by 50 ft., the distance which the author assumes between trucks, the result will be just 800 lb. per lin. ft. Further, if this unit is taken, the excess for a 10-ft. panel will amount to just 24 000 lb., which is a convenient number to use, inasmuch as it is a multiple of both 12 and 10.

If desired, more than one excess load can be used, spaced at any convenient distance apart. In accordance with the author's assumption, suppose, for instance, that heavy trucks will follow each other at distances of about 50 ft., center to center. The excess loads then could be spaced exactly three panels apart for all panel lengths between 15 and 18 ft. with little error. For panels of less than 15 ft., the spacing might be four panels, and for panels of more than 20 ft., two panels. This would result in considerable simplification of the computations.

Simplicity of computation should not be weighed against accuracy of result where the accuracy of the original data is such as to justify a complicated computation; but where the preliminary data may be widely at variance with the actual conditions, a certain degree of approximation to obtain simplicity is entirely proper and reasonable. It is hardly probable that a highway bridge would ever be loaded exactly in accordance with the system suggested by the author, and, therefore, it seems futile to attempt to compute stresses too closely from such a loading. It seems that the results obtained by the "uniform load plus excess" method would certainly give an accuracy in result which would be commensurate with the actual roadway conditions. The resulting bridge would be of properly proportioned design throughout and of sufficient strength to meet all traffic demands for many years to come.

Glenn B. Woodruff,* M. Am. Soc. C. E. (by letter).†—Coming, as it does, at a time when the subject of highway bridge specifications is receiving

* Brooklyn, N. Y.

† Received by the Secretary, September 28, 1923.

so much attention, this paper is most opportune. The simplicity and ease of application of the proposed loadings must commend them to all those engaged in bridge design.

The writer believes that the truck axle loads should be increased to 10 000 and 40 000 lb., respectively, for the following reasons:

1.—In spite of laws to the contrary, truck loadings of 25 tons, or even more, are of frequent occurrence.

2.—A specified load of 25 tons will take care of the stresses produced by two trucks of maximum legal weight in close succession.

3.—Such loading provides some margin for future increase in truck loadings.

4.—In the past, highway bridges have been designed in such manner that it has been necessary to replace them because of a light floor system when the trusses were still amply strong for the heavier loading. The writer doubts whether this feature has been corrected in the recently proposed specifications and believes that the excess weight in the floor, caused by such an increase in loading, would be an excellent investment in any bridge.

The proposed loading diagram might well be extended by adding a uniform load in front of the truck. This change would seem to approach more nearly maximum conditions of loading. It would also simplify stress calculations as the heavy wheel of the truck would then be placed always at the peak of the stress influence diagram.

In selecting loadings for secondary bridges, it should be remembered that the heaviest trucks are likely to be detoured on secondary roads, and provision should be made for such conditions.

The author's suggestion as to the truss load coefficients might be modified to include a percentage of reduction for multiple lines of traffic. The formula, $K = 1 + \frac{W}{20}$, is proposed, in which K equals the coefficient for truss loads and W equals the width of roadway, in feet.

The writer wishes again to express his appreciation of this paper and his hope that, with slight modifications, it may become the standard loading for highway bridges.

THE DISINTEGRATION OF CEMENT IN SEA WATER
Discussion*

By Messrs. E. G. Walker, Emil F. Cykler, Albert Moyer, J. Y. Jewett, Richard Grün, and Jasper O. Draffin.

E. G. Walker,† M. Am. Soc. C. E. (by letter).‡—This paper forms an admirable summary of the major researches conducted during and since the Nineteenth Century, to establish the fundamental chemical facts which govern the setting of cement in sea water. Although the paper contains little that is new, it condenses admirably the work that has been done, and forms a permanent, easily accessible record of that work.

The authors' complaint about the lack of basic experimental evidence on the subject in American technical literature is probably due to the fact that the attention of engineers has been directed largely to the problem of making a concrete that will withstand the action of sea water. The problem presented in the paper forms one (probably the most important) part of this general question, but it is likely to be overlooked in discussion and to be excluded by considerations relating to concrete manufacture, in which it plays no direct part. Realizing the existence of the single standard type of cement, easily obtained, practical engineers have taken this element of the problem as a fixed quantity and in their experiments have endeavored to obtain improved results from a study of the multitude of other variables which enter into the whole operation of making, placing, and hardening of concrete.

For timber construction, engineers select from the many different kinds of trees that most suitable by reason of its physical properties, cost, and durability, and the ease with which it can be obtained. Nature, in this case, has offered man a great variety of materials with which to satisfy his wants. With products requiring elaborate treatment in their manufacture, such as metals, engineers have been led by experience to develop various grades suitable for different requirements; for example, all kinds of ferrous products are used in structural work, from cast iron to special alloy steels, the compositions and treatments of which are designed to fulfill the requirements which arise in practice. The earlier engineers were restricted to the use of cast iron from which they constructed columns, arch ribs, and beams, their designs having been governed by the limitations of its strength and elasticity. Later, wrought iron was adapted to structural requirements, and methods of design underwent a revolutionary change on account of the introduction of riveted work and the development of the truss. From wrought iron, mild steel, and various forms of hard steel

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* Discussion on the paper by William G. Atwood and A. A. Johnson, Members, Am. Soc. C. E., continued from October, 1923, *Proceedings*.

† Westminster, London, S. W. 1, England.

‡ Received by the Secretary, September 7, 1923.

and alloy steel came into use, which, although not adaptable to all requirements, form the most efficient and economical materials for certain special uses. The writer sees an analogy between this and the probable future development in the use of cement.

The authors refer to the famous experiments of Smeaton, in 1756, to find the most suitable material with which to make the mortar for jointing the stones of the Eddystone Lighthouse. His experiments led him to use a material obtained by the burning of the blue Lias limestone found at Aberthaw on the coast of Glamorganshire in South Wales (not in Devonshire, as stated by the authors), which he concluded to be the best hydraulic lime he could procure. The durability of the work proved the wisdom of his selection. The mixture of hydraulic lime and trass used by Smeaton was a cement in the broad sense of the word, but not in the special restricted sense of the present use of the term to indicate a product resulting from intimate mixing, fusion, and fine grinding of the constituents.

About seventy years after Smeaton's experiments, William Aspdin discovered the fundamental fact that by burning a mixture of chalk and clay a material could be obtained which had better hydraulic and cementing properties than the hydraulic limes used prior to that time, and which, as the authors show, had been investigated by Vicat, during and subsequent to 1812. Aspdin's discovery introduced to constructors a new material that differed fundamentally from the old, although in some respects it was in a similar category. During the last hundred years Portland cement has been improved in nearly every way, and a much higher grade of material has been produced by better mixing, more complete burning, finer grinding, and improved methods of handling. Engineers have held, however, to the idea of producing one material for all purposes, yet Smeaton experimented, not because he was unable to find a standard material suitable for jointing stonework, but because he knew that the lime mortars then in use for building in the dry would not withstand the conditions found in sea works. To-day, engineers modify cements in practice to a limited extent to suit special conditions. They make quick or slow-setting cements by adding gypsum in various proportions during manufacture; and they know that different brands of Portland cement have subtle differences of quality which cause one brand to be preferred for one class of work and another for another class, although both conform to the standard specification.

The successive developments of plain concrete, concrete blocks, reinforced concrete, concrete slabs and building tiles, and cement stucco, have introduced the use of cement in a variety of products second only to that of iron. It appears reasonable, therefore, that different purposes can be served best by different kinds of cement. The writer believes that this is a development which will come; the number of raw materials used in the manufacture of cement is increasing. There is already a cement made with blast-furnace slag, alumina cement, and others of different compositions. Unless these materials are to be developed as proprietary articles, it follows that eventually it will

be necessary to formulate standard specifications for different kinds of cement as has been done for different classes of iron and steel. The British Engineering Standards Association has recently issued a standard specification for Portland blast-furnace cement,* which is defined as a cement produced by grinding together Portland cement clinker and blast-furnace slag, the minimum proportion of the former being 35% and the maximum of the latter 65 per cent.

EMIL F. CYKLER,† ASSOC. M. AM. SOC. C. E. (by letter).‡—It is not the writer's object to enter into a discussion of the causes of disintegration of cement in sea water, but rather to describe a certain form of construction which will resist deterioration.

In examining a large number of specimens of concrete such as that used for concrete piling for piers, wharves, and bulkheads, it is evident that the densest concretes contain from 8 to 10% of voids.

The writer has been instrumental in perfecting the following method of treating the concrete used in structures such as sheet-piling or slabs protecting bulkheads:

First.—After the concrete has been allowed to cure for 30 days, the voids are filled with asphaltum, which results in a structure that is water-proof throughout its entire mass.

Second.—The reinforcing steel is coated entirely with asphaltum in place, which prevents the salt water from corroding the bars.

Third.—The compressive strength of the concrete shows a marked increase, some of the specimens testing 75% stronger in compression than the untreated specimens.

Fourth.—A noticeable increase also is noted in the tensile strength of the concrete, as shown when the piling is driven by a steam hammer. The structure is "toughened" to such an extent that no spalling of the head of the pile is noted.

Fifth.—The asphaltum, which is unaffected by sea water, renders the product immune from destructive action of the salt or of frost, inasmuch as it has been water-proofed throughout its entire mass.

Although Los Angeles Harbor is not subject to freezing, all permanent structures are being founded on piling treated with asphaltum.

A concrete has been developed, therefore, which should have an indefinite life. It is the only product which can be classed properly as a permanent structure when used in sea water. This product has been named "Duocrete," that is, a "double" concrete, consisting of a Portland cement concrete and an asphaltic concrete. It was developed to resist various destructive agencies, as pointed out in the discussion on the disintegration of concrete in sea water. The writer submits this information for the benefit of members, hoping that the new material may overcome many of the present difficulties of water-front construction.

* *Publication No. 146* (1923), British Engineering Standards Association.

† Gen. Mgr., Pan-Pacific Constr. Co., Los Angeles, Calif.

‡ Received by the Secretary, September 18, 1923.

ALBERT MOYER,* AFFILIATE, AM. SOC. C. E. (by letter).†—This discussion is divided into a summary, an argument, and, finally, a statement of the problem to be investigated.

Summary.—The writer will attempt to show by this discussion that there is little foundation for the assumption that well placed dense concrete, composed of the best Portland cement and clean aggregate, is decomposed (chemically) by sea water either between tides or below low tide. The references mentioned by the authors seem to apply to investigations of the decomposition of concrete rather than to the disintegrating action. There is a vast difference between decomposition and disintegration; one is entirely chemical and the other is entirely mechanical, and, yet, in the Synopsis of the paper, the authors speak of "disintegration of concrete in sea water", failing to differentiate between the two actions. In the Introduction it is stated that "it appears to be reasonably certain that, if there is no chemical cause, serious failures of properly built structures need not be expected." The writer will attempt to show that disintegration (mechanical action) is the cause of a very large proportion of the deterioration of concrete in sea water, whether the cement is high alumina, high silica, or whatever it is, as long as it is of good quality.

In the following discussion a serious suggestion is offered that a thorough investigation be made of samples of concrete (both below tide and between tides) that have remained intact, and in satisfactory service for ten years or more. Having determined the cause of the success in these cases, engineers and others should formulate specifications and practical suggestions for the benefit of those who are about to undertake construction with concrete in sea water. The writer refers to concrete made of standard Portland cement, sand, and gravel or crushed stone, without any admixture whatsoever, and, furthermore, concrete is considered as a rock, similar to that produced by Nature.

Argument.—A scientific problem is treated scientifically only if it leads toward a practical solution. The erosion, decomposition, and disintegration, of rock (or concrete) is caused by various agencies. There are both chemical and mechanical methods of breaking up rocks into particles or of dissolving them. When rocks are broken up chemically it is called "decomposition" and the product of decomposition differs from the original. The question arises, when concrete is eroded or disintegrated between tides, "Does the product of this destruction differ from the original material?" The mechanical method is merely a separation of the different minerals into their component parts, called "disintegration". When both decomposition and disintegration act simultaneously, under the influence of water vapor and gases, with accompanying temperature changes, the process is called "weathering". This latter action can hardly be the cause of deterioration of concrete between tides, as the required water vapor and gases accompanied with high temperature are not present.

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† Received by the Secretary, September 20, 1923.

Artificial or natural rocks, as a whole, are poor conductors of heat and, therefore, the surface layers are subjected to an expansive force, while the deeper layers are unaffected. It has been noted also that the wetting of concrete or natural rocks expands the surface and that drying contracts it. Therefore, between tides, the surface of rock or concrete is always slightly in motion. This is one of the principal causes of hair cracks and "crazing" of concrete surfaces. The two actions, heat and water, might have some slight influence on the disintegration of the surface of concrete between tides. Various natural rocks and the aggregates composing concrete re-act differently to heat and cold, and, therefore, surface stresses in these rocks or aggregates are set up when temperature changes occur between tides, which might tend to separate one mineral from another. This is particularly noted in granite. The slopes of Pike's Peak, composed of coarse granite, are covered with such disintegration products. Finally, it is noted that minerals themselves are affected by these temperature changes, because minerals with several distinct crystal axes re-act differently on the various faces. Thus, a crystal of feldspar is itself subjected to internal stresses which open minute fractures along the cleavage line. This is how kaolin is formed.

Disintegration can be caused by the expansion of ice crystals. Water on freezing increases one-tenth in volume, and thus becomes a powerful agent in disrupting rock. Very rapid disintegration might occur to natural rocks if they were porous, or in concrete. If, however, the rock or the concrete were very dense and strong, it is not likely that frost action would have an opportunity to do its work of disintegration. The effect of frost on the soil is one of expansion, by filling the pores with ice crystals, which lift and separate the particles. This may be noted on a frosty morning. When the ice crystals have melted, the soil is left full of small cavities which give the mass a loose and spongy appearance much like that of porous concrete that has been similarly affected by the growth of ice crystals in the pores between tides. Similar action is occasioned by the growth of salt crystals. The following is quoted from the conclusions of the U. S. Bureau of Standards:

"Portland cement mortar, or concrete, if porous can be disintegrated by the mechanical forces exerted by the crystallization of almost any salt in its pores, if a sufficient amount of it is permitted to accumulate and a rapid formation of crystals is brought about by drying, and as larger crystals are formed there would be obtained the same results on a larger scale. Porous stone, brick, and other structural materials are disintegrated in the same manner."

The following quotation is from the Encyclopedia Brittanica*:

"Good hydraulic cements are highly permanent materials provided certain conditions be observed. It might be supposed that hydraulic cements from their nature would be indifferent to the action of water, but this is only true if the structures of which they form a part are sufficiently compact. In this case the action of water is checked by the film of carbonate of lime, which eventually forms on the surface of calcareous cement. This together with the compactness of the mortar, hinders the ingress and egress of water and prevents the dissolution and ultimate destruction of the cement. But where the concrete or mortar is not well made and is porous, the continual passage of water

* Eleventh Edition, Vol. 5, p. 658-B.

through it will gradually break up and dissolve away the calcareous constituents of the cement until its strength is utterly destroyed. This destructive action is increased if the water contains sulphates or magnesium salts, both of which act chemically on the calcareous constituents of the cement. As sea water contains both sulphates and magnesium salts it is especially necessary in concrete for harbor work to take every care to produce an impervious structure."

There is a preponderance of evidence that nearly all concrete that has deteriorated in sea water is situated between tides. The submerged concrete constantly under water is seldom affected by chemical or mechanical action of the sea water. It is, therefore, reasonable and logical to assume that disintegration is due almost entirely to the causes mentioned. Denser rock is affected much more slowly than porous rock, which accounts for the so-called "everlasting hills", composed of the densest rock, which was formed in pre-Cambrian times.

The Problem.—The writer believes that it is futile to carry on exhaustive research work in regard to the chemical cause of the decomposition of porous concrete, but that thorough investigation, based on existing successful structures which have been immersed in sea water or between tides for ten years or more, can lead to conclusions of the greatest value.

From the numerous examples of such construction, undoubtedly information may be obtained, giving the specifications, the kind of aggregates, the character of the Portland cement, the proportion and, possibly, the methods of mixing and of placing. This will furnish the nucleus for research work, leading to proper recommendations. One of the best references on this subject is a report of a Special Committee, appointed by the Institution of Civil Engineers of Great Britain to study the deterioration of timber, metal, and concrete, when exposed to the action of sea water. This report* was edited by Messrs. B. M. Crosthwaite and C. R. Redgrave, and is a compilation of reports of various members of the Institution on the condition of structures in all parts of the British Empire, written for the most part by members who are officially connected with the operations of the various ports. In general, their attitude toward concrete is very favorable, and they seem to have had almost uniformly successful results in some cases for more than 10 years. The first report was issued in 1920 and a second in the latter part of 1922, or the early part of 1923.

In the report of Committee E-6 of the American Concrete Institute presented at the Annual Meeting of the Institute at Cincinnati, Ohio, in January, 1923, there appears a table (Table 2), on the Aberthaw tests of concrete in sea water, carried on at the Charlestown Navy Yard, Boston, Mass. This table certainly bears out the writer's argument. The dense concrete stood up well regardless of the cement used.

In the investigation and research work which, it is hoped, will follow this discussion, it is respectfully suggested that in examining concrete that has decomposed (that is, where it is assumed that chemical action has taken place), the sediment which may remain in the pores be analyzed to determine

* Published by the Committee at Great George Street, Westminster, S. W. 1, England.

whether or not it differs from the material of the original concrete. Unless it differs, it shows disintegration or mechanical action and not decomposition.

To repeat, the problem is to ascertain why the numerous and prominent examples of concrete have withstood all these alleged chemical and mechanical actions and are still performing their useful functions although immersed in sea water for periods of more than ten years.

J. Y. Jewett,* Assoc. M. Am. Soc. C. E. (by letter).†—The authors state‡ that they have been unable to find any record of tests in the United States on pozzuolanic materials with relation to resistance to the action of sulphate-bearing waters. Reference is made to the use of "blended" cements of this type, as represented by the sand cements used in certain dams built by the U. S. Reclamation Service, and the tufa cement used on the Los Angeles Aqueduct, and surprise is expressed that no sea-water or alkali-soil tests of these cements were made. Although no results were published, a set of such tests, under alkali-soil conditions, was made by the writer about ten years ago, during his period of service as Cement Expert in the U. S. Reclamation Service.

A review of the experience of the U. S. Reclamation Service in dealing with the problem of alkali action on concrete may be of interest. When this trouble first appeared on certain projects in 1908, it became evident that a study of the subject was needed. The officials of the Service, felt that Reclamation funds could not be used for such purposes, and asked the Structural Materials Laboratory of the U. S. Geological Survey (later transferred to the U. S. Bureau of Standards) to consider the matter. This led to the inclusion of a study at Atlantic City, N. J., by this laboratory, of alkali action with one of sea water action, the results of which have been published.§

In the meantime, the writer made some field experiments with the limited facilities at his command, and with such aid as could be obtained from the engineers of the projects affected. At first, these experiments were conducted on small specimens made in the laboratory and sent to the various projects for exposure where alkali action was most severe. Later, a series of tests was started on specimens made in the field, using, in part, materials sent out from the laboratory, and, in part, materials available on the projects themselves. The materials used included sand cements manufactured by the Service, and also a so-called "alkali-proof" cement of the same general type from one of the cement companies. In the latter cement, blast furnace slag was the siliceous material used in the blending process. The series also included samples of various other materials and processes, for which claims of waterproofing properties had been made. The specimens were in the form of blocks, about 24 in. by 9 in. by 12 in., and when exposed for test, were placed on end, about one-half being above ground line or water line.

Observations of the action on these specimens showed that a rich dense mixture of Portland cement concrete was more resistant to the alkali action

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† Received by the Secretary, September 4, 1923.

‡ *Proceedings*, Am. Soc. C. E., August, 1923, p. 1053.

§ *Technologic Paper No. 12*, U. S. Bureau of Standards.

than that of any other material used. The writer felt, however, that the series was not sufficiently comprehensive to be conclusive, as it was difficult to get the attention to details so essential in carrying out the series as a whole. When the writer urged the need of co-ordination by the employment of an assistant whose duty would be that of conducting a field series on a broad scale, keeping detailed records of all factors having a bearing on the problem, he was again met with the limitation of lack of funds for work of this type.

Arrangements were made, however, with the U. S. Bureau of Standards to add to its work on drainage tile a series of tests on mass concrete. The results of both these series have been published in the *Technologic Bulletins* of that Bureau. The Committee which outlined the program for this series of tests confined them to straight Portland cement concrete. As previously stated, the results obtained on the series carried out by the writer were not published, but they were available to this Committee, and may have influenced its decision to restrict its tests to various proportions of mix and types of aggregates, using only Portland cement concrete.

One point of importance in the use of these blended cements is that of their durability under general climatic conditions, aside from the special service conditions under discussion. Experience has shown that the sand cements of the Reclamation Service, while excellent for use in mass concrete in a large dam where not exposed to surface wear or weathering, are not as durable as Portland cement where thus exposed. In this connection, it developed that laboratory tests did not fully indicate field service qualities. Materials showing high strength in laboratory tests developed poor wearing qualities and lack of resistance to frost action, in the field. This was markedly true of the previously mentioned "alkali-proof" cement.

Granting the soundness of the theory on which is based the proposed use of cements of this type for resistance to sulphate-bearing waters, it would seem, therefore, that in the experiments recommended by the authors, one of the main features should be the selection for blending of those siliceous materials which will prove not only resistant to the action of these waters, but also to frost action and other climatic effects.

Incidentally, it is of interest to note that the term, "sulphate-bearing waters," is coming into use in the discussion of the action of both sea water and alkali on concrete. It became evident in the early stages of the investigation of alkali action, that the sulphate salts were the principal cause of disintegration. Therefore, the term, "sulphate-bearing," is more suitable technically than the general term, "alkali", which has been so commonly used in this connection.

As a matter of correction, also, it may be noted that, although sand cement was used in the Arrowrock and Elephant Butte Dams of the Reclamation Service, it was not used in the Shoshone Dam, as stated by the authors. The Shoshone Dam was built at an earlier period than the other two dams, at a time when investigation on sand cement had not reached a stage where its use seemed justified.

RICHARD GRÜN,* ESQ. (by letter).†—The writer has studied this paper with great interest, and notes that the extended use of blast-furnace cement, as it has been carried on in Germany for the past ten years, appears to be incompletely understood in America. Attention also is called to the fact that the chief research worker in blast-furnace cement, Dr. Hermann Passow, has not been mentioned in the paper, and, therefore, he must be also unknown.

Dr. Passow has written several papers on the use of blast-furnace cement in sulphate-bearing waters, which show that blast-furnace slag is one of the best known pozzuollanas for the improvement of Portland cement for resistance to salt water. The best proof of the unusual serviceability of blast-furnace cement is the fact that, in many of the German blast furnaces, blast-furnace slag is converted into a cement that is markedly cheaper than Portland cement and has a much greater resistance to salt water. The writer has been for many years the Director of a manufacturing plant for making blast-furnace cement and, on this account, is in a position to judge of its great importance.

Attention is called to the "Handbuch für Eisenbeton",‡ in which the influence of various liquids on cement is fully discussed. In Germany, blast-furnace cement, that is, a mixture of 70% of blast-furnace slag and 30% of clinker, often is used in place of Portland cement to withstand the action of salt water, and, in addition, many experiments have been carried out, which prove its value.

JASPER O. DRAFFIN,§ ASSOC. M. AM. SOC. C. E. (by letter).¶—This paper presents an excellent résumé of the literature on the effect of salt water on concrete and cement. It is particularly valuable in that it gives such a complete bibliography of the work done and the theories evolved by engineers and investigators.

The writer has had no experience with concrete in salt water and, therefore, cannot contribute directly to the subject discussed. He desires, however, to call attention to the fact that the paper, by its general tone, emphasizes the need for more knowledge of the reactions that occur during and after hydration, of the properties of the compounds formed, and of their actions on each other.

It is generally assumed, and often stated, that iron and magnesia are negligible constituents in the cement and that they do not affect the formation of the strength-giving compounds. This may be true for most cases and yet not be entirely true in certain special cases. It is also assumed that the aggregate remains unchanged during and subsequent to hydration; again, this may be generally true, but the writer knows of tests of material for construction purposes in which the only apparent explanation of the test results was that a chemical reaction between the cement and the fine aggregate had occurred. The statement by Chandler Davis,‖ M. Am. Soc. C. E., that precast concrete blocks resist the action of sea water better than the cast-in-place

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† Received by the Secretary, September 26, 1923.

‡ Third Edition, Vol. 5, pp. 31–71.

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‖ Received by the Secretary, October 6, 1923.

¶ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1659.

blocks, points to a certain inhibition by the salt water of normal hydration of the cement in the cast-in-place concrete. There may be no such inhibition, but the fact that such a variation exists in the quality of the two kinds of blocks is evidence that the action is not the same in each case. If there is a variation in the results obtained under slightly different circumstances, there should be careful consideration of all possible factors that may affect the results.

These different points are mentioned merely to call attention to the need of examining some of the assumptions. It must be remembered that, as the knowledge of the subject increases, the necessity for more exact information also increases. To quote from the discussion* by Benjamin A. Howes, M. Am. Soc. C. E., "* * * the permanence of Portland cement and the concrete made from it calls for a more scientific knowledge of the constitution of the cement and its hydration * * *." The writer concurs with Mr. Howes in this view and believes that considerable advance will be made in the manufacture of cement and concrete by applying the same general principles that have been so well applied in recent years by metallurgists, petrographers, and physical chemists in their respective fields. This should be especially true as regards the use of the microscope in the study of cement and concrete.

* *Proceedings*, Am. Soc. C. E., September, 1923, p. 1651.

IMPROVED TYPE OF MULTIPLE-ARCH DAMS

Discussion*

By Messrs. H. de B. Parsons and Herman Schorer.

H. de B. Parsons,† M. Am. Soc. C. E. (by letter).‡—The author's favorable comments on multiple-arch design for dams are well founded, but such dams require careful designing and detailing.

It goes without saying that the foundation material under any dam should be impervious, but, unfortunately, there are sites where this condition does not exist naturally and cannot be created absolutely by man. In such cases, the engineer must try to prevent the water that flows under the dam from obtaining velocities which will wash out the foundation material. Generally speaking, the longer a dam stands on such pervious material, the safer it will become, by virtue of the silting up of the bed of the pond behind the dam.

Where the foundation is rock, the multiple-arch dam is probably the most economical type; where it is not rock, probably the multiple-arch principle is the safest on which to design dams. The spacing of the buttresses where the foundation is compressible, should not be too wide, as narrow spacing will make a lighter and more nearly uniform loading on the material under the base. Reasonably short spans for the arch barrels have an advantage in form work; the forms can be used again, and are lighter and more easily handled by a cableway than long-span forms. This construction feature should be considered in determining the span length between buttresses.

The slope of the water face should be made flatter for multiple-arch dams on compressible foundations than for those on rock. A flat slope, that is, less than 40° with the horizontal, permits the weight of the water to augment the weight of the concrete, thus increasing the resistance to sliding of the base on the sand or other compressible foundation material. One dam about 70 ft. high, designed by the writer, had a slope of the arch barrel of 23° to the horizontal.

No matter how the stresses in a multiple-arch dam are calculated, there is always doubt regarding their accuracy. Buttresses should be erected first and allowed to settle, and the concrete should be allowed to shrink, before the barrel arches are poured. This is especially true for dams on compressible material. The writer feels that the construction joints at the top of the buttresses should be close to the vertical sides of the buttresses. In other words, the side of a buttress below the construction joint should not curve out on the intrados of the arch any more than is absolutely necessary.

* Discussion on the paper by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., continued from October, 1923, *Proceedings*.

† Cons. Engr.; Prof. Emeritus, Rensselaer Polytechnic Inst., Troy, New York.

‡ Received by the Secretary, September 24, 1923.

The double wall buttress illustrated by the author has merit in facilitating stress calculations and avoiding struts, but is expensive in form work and difficult to inspect. For low dams, it would be uneconomical, although for dams of more than 125 ft., such a design might be worth studying. Should there be uneven settlement of the dam base under buttresses on compressible material, both bending and shearing stresses will occur in the web walls between the buttresses, which stresses might be destructive.

Fig. 25.

The small arch on the water side between the double buttress walls will be considerably more rigid than the main arches, and temperature changes will set up stresses that are not balanced on each buttress side-wall. Instead of having double wall buttresses, attention could be given to a buttress design with counterforts for stiffening, as shown on Fig. 25, where the buttress has parallel sides or is widened toward the downstream toe of the dam. This latter form has the merit of maintaining a more uniform loading on the foundation. Carrying this idea further, the counterforts could be made to be more than stiffening pilasters and to carry the thrust of an arch should part of the dam fail, a condition suggested by the author as favorable to double buttresses. This idea, shown in Fig. 26, was suggested by the writer's Assistant, Mr. Vincent J. McKinnon.

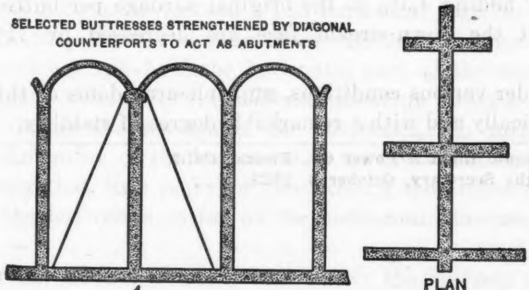

Fig. 26.

Seasonal temperature changes of the concrete are due to water and air. The air temperature in the bays under the arches will be lower on hot summer days and higher on cold winter days than the outside temperature. The temperature of the intrados of an arch will approach that of the air with which it is in contact, while the temperature of the extrados will be that of the water. Therefore, the concrete will not be at a uniform temperature throughout the thickness of the arch in the extreme seasons. As most concrete will be found to be somewhat porous, the water temperature will largely govern, and the mass will be chiefly at the temperature of the water, while a minor thickness at the intrados will approach that of the air. The buttress concrete,

however, will be affected by air temperature changes and not by water temperature. Being buried in the ground, the base of the dam will have only a relatively small temperature range. It is fortunate that the capillary action of water in concrete, that is, the internal dampness of the concrete, will tend to diminish sudden changes of temperature throughout the concrete, as, for instance, where the barrels of the arches project from the pond and also where they join the buttresses.

HERMAN SCHORER,* ESQ. (by letter).†—After reading this paper, the writer was sufficiently interested to investigate what change in the maximum stresses might result from altering the down-stream slope of the buttresses from 1 : 10 to 1.5 : 10, all other dimensions being the same as those given by the author. The quantities of a buttress are thereby increased by about 4.5 per cent.

By using the monolithic theory, a comparison of the principal items is given in Table 1.

TABLE 1.

Batter.	Neutral axis, in feet.	Stresses, in Pounds per Square Inch.	
		f_1.	f_1 max.
1 in 10	34.7	468	473
1.5 in 10	58.7	417	426

Therefore, by adding 4.5% to the original yardage per buttress, the maximum stresses at the down-stream face are decreased by 11% and 10%, respectively.

No doubt, under various conditions, multiple-arch dams of this type can be designed economically and with a remarkable degree of stability.

* With San Joaquin Light & Power Co., Fresno, Calif.

† Received by the Secretary, October 9, 1923.

STRESSES IN MULTIPLE-ARCH DAMS

Discussion*

BY LARS R. JORGENSEN, M. AM. SOC. C. E.

LARS R. JORGENSEN,† M. AM. SOC. C. E. (by letter).‡—The author gives an elaborate analysis of the stresses, and their distribution, in a buttress of a multiple-arch dam.

The economical height of a multiple-arch dam is controlled by the design of the buttresses, which require much more material than the arches for high structures. With more accurate formulas available for calculations, higher stresses can be used with safety. Naturally, this will influence economical design.

Although apparently it might be desirable to build the buttress of a high multiple-arch dam with a vertical up-stream face, such a dam would always have an uncertain factor of safety as it would have to depend too much on the shearing strength of the buttress material to hold the structure in place.

A high buttress cannot be poured in one operation, nor can the necessary construction joints be made vertical. The horizontal part, also, has an uncertain bond strength. With a sloping up-stream face, the vertical component of the water pressure will hold the horizontal part of the construction joints together, and make them safer for moderate shearing stresses.

When calculating the stresses in the buttresses, the author includes the arches in the dimension of the buttresses in an up-stream and a down-stream direction, stating that, in a paper by the writer,§ the arches are not assumed to be part of the buttresses as far as the horizontal dimension is concerned, which is true.

The supporting surface is more than merely the buttress proper, but it is somewhat a matter of judgment as to how much more than the actual buttress should be included in the length of the contact surface for the purpose of calculating actual foundation stresses. It appears to the writer that to include the whole arch is extreme and that midway between the crown and the groin would seem to be more logical.

The author suggests that the theory, as given in paper "The Circular Arch Under Normal Load",‖ by William Cain, M. Am. Soc. C. E., be used for the calculation of the arches. For arches as thin as those used in multiple-arch dams, in which the stresses along both faces are about equal, these formulas

* Discussion on the paper by B. F. Jakobsen, M. Am. Soc. C. E., continued from October, 1923, *Proceedings*.

† Cons. Engr., San Francisco, Calif.

‡ Received by the Secretary, September 17, 1923.

§ "Multiple-Arch Dams on Rush Creek, California," *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 850.

‖ *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

are the most accurate available. For heavy single arches, however, these or any other formulas known to the writer, must be used with discrimination, as they do not take into consideration several factors which tend to modify the stresses.

For instance, if Professor Cain's formulas are applied to the Lake Spaulding Dam on the Yuba River, California, at Elevation 4740, where the arch is 58 ft. thick, and the water pressure is that due to a 135-ft. head, the stresses at the abutments are apparently 405 lb. per sq. in. tension at the extrados and 865 lb. per sq. in. compression at the intrados.

If the tension ever approximated 405 lb. per sq. in., the concrete would crack vertically, but no such crack or cracks are in evidence, although near the up-stream face, there are vertical drain pipes spaced 4 ft. apart, which would facilitate and invite the formation of vertical cracks. Therefore, little or no tension can actually exist along the up-stream face at the abutments and, consequently, the unit compression along the down-stream face is correspondingly lower than the calculated value of 865 lb. per sq. in.

There are at least two causes for this redistribution of stress, one of which, as pointed out by Mr. Jakobsen, is the swelling of the concrete along the up-stream face or water-side, relieving the tension; the second cause is the influence of the time factor on the modulus of elasticity. Should the stress in the concrete material ever tend to reach such values as those previously mentioned, a redistribution would automatically take place, due to the influence of this time factor, and that is what has actually happened.

The author uses an arch which is circular in a plane perpendicular to the up-stream face of the buttress. The writer agrees with this, as he believes that the formwork not only will be less expensive thereby, but also that it will be more accurate than if the arch is made circular in a horizontal plane, which would require an elliptical truss for supporting the face forms.

The arch should be built of uniform thickness in any horizontal plane, as the water pressure is the same at every point in such a plane. By using different lengths of spreaders between the up-stream and the down-stream face forms, this is easily accomplished, the up-stream face remaining cylindrical.

THE RIVER AND HARBOR PROBLEMS OF THE LOWER MISSISSIPPI

A SYMPOSIUM

Discussion*

By E. J. Dent, M. Am. Soc. C. E.

E. J. Dent,† M. Am. Soc. C. E. (by letter).‡—In the course of his training, and later in connection with his practice of river and harbor engineering, the writer has often seen the need of better hydraulic training for the engineers engaged in this work. Many graduates of the best esteemed universities in the United States are very deficient in their knowledge of the practical application of hydraulic laws when they first encounter the problems of the great American rivers. A few minutes spent observing a Venturi meter, followed by the actual computation of the discharge, will teach the average student more about velocity head than he could learn by hours of theoretical discussion. What is still more important is that, with a practical application of the principle firmly fixed in his mind, the embryo engineer is ready to apply it to some other problem as soon as it arises. The same rule applies to the flow of water over weirs or through orifices. For teaching the effect of roughness of sides of channels or pipes, for illustrating the readjustment of flow in elbows and bends, and similar phenomena, the writer would like to see the use of open flumes more generally adopted. In an open flume, the student could actually see the turbulent flow, the eddies, and the boils, and the effect of such optical demonstrations would be most beneficial. In many parts of the Mississippi Valley, and in some other parts of the country, a current meter is nearly as important a part of the equipment of a civil engineer as a transit, and the student should be taught how to use it in the field and how to apply the results to the practical problems of design.

In water-works, drainage, irrigation, and river work, the transportation of sediment by running water and its deposition whenever the current slackens to below the critical velocity, are of such prime importance that special courses on these subjects may often be justified. In highway and railroad work, erosion is a matter of prime importance to the engineer, the tax-payer, and the capitalist. In hydro-electric work, the problems of flowing water are numerous and complex.

It seems to the writer that a well equipped laboratory, including both a weir flume and a river flume, should be an essential feature of the equipment of every great university. The statement of the case by Col. Millis, in his

* Continued from October, 1923, *Proceedings*.

† Lt. Col., Corps of Engrs., U. S. A., New Orleans, La.

‡ Received by the Secretary, September 7, 1923.

discussion,* deserves the support of all engineers. The question whether, in addition to the university laboratories, there is need for a great National laboratory, is one that cannot be so readily answered. On this subject, the writer reserves judgment until the facts have been presented in a more judicial manner than has been done by Mr. Freeman and until the problems listed for solution in such a laboratory have been more carefully considered. The idea that for problems of river improvement, the little rivulets, flowing through the river flumes of the laboratories, can be treated as miniature of the actual rivers, is essentially unsound, and the fact that most river engineers agree with this idea is no proof that the Profession has slumbered for seventy years, or that it is still slumbering.

Mr. Freeman repeats a statement made by himself before a Committee of Congress, namely,

"* * * that the steamboat pilot on the river and the scientist in his laboratory each has certain advantages in his point of view, and that much of the river-training work in America has been of that quality that might be expected to be produced by a committee of steamboat pilots without special training in exact science, * * *."

The Annual Reports of the Mississippi River Commission since its creation in 1879, are available in many engineering and public libraries. If any member of the Society will look into these reports, he will find the names of an astonishing number of engineers who have risen to National and international fame: Presidents of the Society; Chiefs of Engineers, U. S. Army; educators; inventors; consulting engineers; and, finally, the rank and file of those men who by hard work and common sense have earned the esteem and respect of all who have had the privilege of knowing them. The members of the Society may rest assured that the methods devised by these men rest on a sound scientific as well as on a firm practical foundation.

The idea that small-scale models may be used to develop the best form of structures for the improvement of waterways is not new. During the past few years, it has been repeatedly suggested to the writer that he use this method for the solution of certain of his problems. None of those making the suggestions has volunteered to work out the details; some have frankly stated that they considered the writer the proper person to do that part of the work.

To resort to a model for the solution of any particular hydraulic problem is, in reality, to abandon science and fall back on empirical methods. The writer knows very well that experience, when available, is a better guide than imperfect theories, but, like other engineers, he also knows that empirical laws cannot be safely extended far beyond the limits of the experiments from which they were derived. It is perhaps unfortunate that empirical data are so often expressed in the form of algebraic equations which can be easily extended to the limits of the imagination by any youthful student of mathematics. Such equations form a fruitful field for engineering blunders. Data plotted in the form of diagrams are not so readily extended and are, therefore, not so likely to be abused.

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1262.

Mr. Freeman has suggested* a model of the Lower Mississippi River on a horizontal scale of 1 : 1 000 and a vertical scale of 1 : 100 or 1 : 1 000. Few people are able to visualize the relative sizes of such a river and such a model, and few are able to visualize the absolute size of this mighty stream. To aid in the mental comparison of a section of the river with something more familiar, Fig. 44 has been prepared. The upper part of this diagram shows

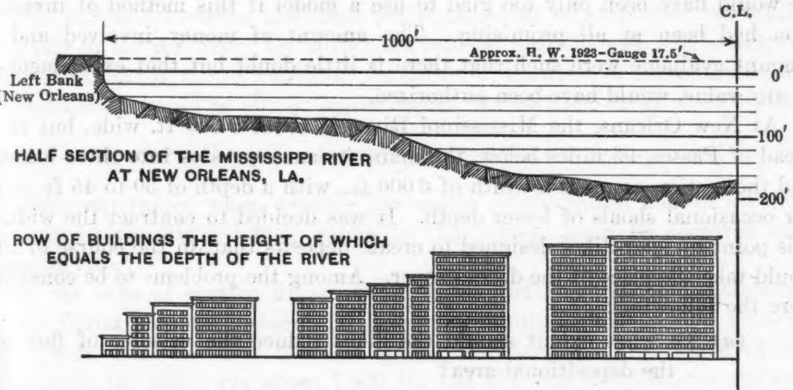

FIG. 44.

in true scale a half-section of the Mississippi River within the limits of the City of New Orleans. Immediately below this half-section, there has been drawn a group of buildings of substantially the same height as the depth of the river immediately above. A model of this half-section built on a scale of 1:1 000 would be 1 ft. long and 0.20 ft. high. Let the reader compare this with the size of 1½ bricks in the face of one of the buildings. If the vertical scale were 1 : 100, the length would be the same, but the height would be increased to 2 ft. Mr. Freeman bridges this gap in size by the following statement:

"At first, one feels misgivings about the small dimensions used by Professor Engels in his model of the flowing stream giving correct relations for the real river, which may be a hundred or perhaps a thousand times the linear dimensions of the model. Still, ordinary formulas for canals, pipes, weirs, and orifices, are substantially correct through a similarly wide range, and Professor Engels and many others state that the correspondence of behavior between model and original has been proved satisfactorily in many cases."

In river engineering, the ordinary problems of flowing water are complicated by such items as suspended sediment, sand dragged along the bottom, caving banks, snags, and drift. Nowhere in his paper does the author acknowledge that under these conditions a model on a scale of 1 : 100 or 1 : 1 000 will prove unreliable, but when it comes to a problem of clear water flowing over a fixed weir, he considers it necessary to extend the work of past investigators on account of the pitifully small scale on which the experiments were conducted. The late James B. Francis, Past-President, Am. Soc. C. E., used depths up to about 2 ft. in his classical weir experiments. Surely it would be safer to use

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1196.

his formula for weirs having an overflow depth of 20 ft., than to design river-control structures by comparison with models only $\frac{1}{100}$ or $\frac{1}{1000}$ the size of the real river.

In using models to design river-control works, size is not the only difficulty; in these models there are distortions in relative values that should not be ignored. To illustrate this phase, the writer will cite an actual case in which he would have been only too glad to use a model if this method of investigation had been at all promising. The amount of money involved and the amount available were such that there is little doubt but that experiments, if of any value, would have been authorized.

At New Orleans, the Mississippi River is about 2 000 ft. wide, but at the Head of Passes, 95 miles below, the main stream separates into three branches and the section assumes a width of 6 000 ft., with a depth of 30 to 45 ft. except for occasional shoals of lesser depth. It was decided to contract the width at this point by spur-dikes designed to create deposits that, in the course of time, would take the place of the dikes proper. Among the problems to be considered were the following:

(*a*) To what extent should the dikes reduce the velocity of flow over the depositional area?

(*b*) How close together should the dikes be built?

(*c*) What was the most economical form of dike to use?

It might seem that Items (*a*) and (*b*) were typical questions to solve by means of a laboratory model, but, before finally adopting this point of view, the details would deserve study.

The annual flood in this locality is the result of a combination of numerous minor floods. No two floods are alike in their magnitude or in the character of the sediment carried. No exactly scientific solution can be obtained in any event. The sediment borne by the river consists mainly of very fine sand and clay carried in full suspension. On an average about 40% may be considered as sufficiently coarse to be available for forming deposits below the dikes and about 3% additional may be considered as being transported by what Gilbert describes as the "saltation" process. The remaining 57% would be so fine that no feasible way of causing it to deposit could be found.

If a laboratory model were built on a horizontal scale of 1 : 1 000, the width would be about 6 ft., which would be feasible with the proposed laboratory, and the length of the proposed depositional area would be 7.5 ft. The question of the proper depth deserves some consideration, as the depth in the river is only 30 to 45 ft. If a scale of 1 : 100 were used, the depth in the model would be only a few inches. For the sake of discussion, a vertical scale of 1 : 30 will be assumed.

If, in the model, real water and real sediment are used, it follows that there must be real velocities to carry that sediment. The bottom velocity in the model must be substantially the same as that in the river; otherwise the load of sediment moved by the "saltation" process would be very different in the two cases, and, if the velocity in the model is too low, the sediment in suspension will soon be deposited. Perhaps the best velocity to adopt is that of

the bottom foot of water in the real river, which for this case may be assumed as 3 ft. per sec.

With a velocity of 3 ft. per sec. in the model, the bottom load per unit of width will be the same as for the main river or three units. In the real river, there are above this bottom load 40 units of suspended material sufficiently coarse to be potentially available for the formation of deposits. If the water used in the model carries the same percentage load of suspended material as the water in the real river, the suspended load per unit of width will be only one-thirtieth of that in the real stream. In the river, the ratio of suspended heavies to bottom load would be 40 : 3; in the model, the corresponding ratio would be $\frac{40}{30}$: 3. In any river, the ratio of suspended sediment to bottom load becomes of great importance when deposits are under consideration and in all model experiments the distortion of values in this respect must be accepted as an inherent vice.

In the case of certain dikes already built and functioning satisfactorily, it was found that the velocity of the water in the sedimentation area was about 3 ft. per sec. For the case under consideration, the length of bank affected by the dikes was about 7 500 ft., and the time required for the water to traverse this distance would be 2 500 sec., or nearly 42 min. In the case of the model, if the velocity below the dikes is $1\frac{1}{2}$ ft. per sec., the time required to traverse the 7.5 ft. of laboratory bank would be 5 sec. In the real case, a particle of sediment starting from mid-depth must settle about 15 ft. in 42 min. in order to be deposited. In the laboratory model, a particle starting at mid-depth must settle 0.5 ft. in 5 sec. Thus, in the real river, the rate of settlement would be 0.36 ft. per min. and in the model it would be 6.02 ft. per min.

In that part of the river between the ends of the dikes and the opposite shore, the mean velocity in the real river is about 6 ft. per sec.; in the model, about 3 ft. per sec. would be the proper velocity, as the model stream represents bottom conditions more accurately than any other conditions. The velocity in the model, therefore, would simulate a velocity of 3 000 ft. per sec. in the real river, this being about the speed with which a bullet leaves the muzzle of a modern army rifle. If under these conditions the model shows that sand picked up at one point will be deposited at some other point, it is by no means established that the same would be true of the real river.

At the time the design of these dikes was under consideration, the writer was unable to understand how laboratory models could have helped to arrive at a satisfactory solution, and he is still of the same mind in this respect.

The determination of the best form of structure to be used involved consideration of the ordinary, practical, every-day problems of the engineer-contractor. The depth of water varied from 3 ft. near the shore to more than 35 ft.; the currents were 0 to 4 miles per hour; the upper dike must stand against accumulations of acres of drift, and the outer ends of the other dikes must withstand some drift; the bottom would vary in a few feet from soft mud to compact fine sand; the outer ends of the dikes must be pro-

tected from scour; and the life of the dike must be about 5 years. It is obvious that these problems could not be solved in a laboratory. Mr. Freeman has shown* that 40 years ago in the Plum Point Reach, the Mississippi River Commission found that the real problem was to design an economical structure strong enough to resist the pressure of the drift that piled up 20 ft. high and the currents that underscoured the dikes. These were not laboratory problems. Woods Brothers did not perfect their method of sinking anchor piles by laboratory experiments on a scale of 1 : 1 000.

Hydraulic laboratories on a modest scale in all the universities would soon develop a force of graduate engineers who would know what information to seek from the full-scale phenomena of real rivers and how to record the observations. The influence of the Society should be directed toward developing these practical students rather than toward the construction at this time of a great National laboratory, such as that advocated by Mr. Freeman.

In addition to his discussion of a National Hydraulic Laboratory, Mr. Freeman discusses some of the features of river improvement work. Although a few of these features are mentioned subsequently, no attempt has been made to cover all the questions raised.

Having seen spur-dikes or groins successfully used on certain streams for guiding the current and preventing bank erosion, Mr. Freeman suggests that they be used on the Mississippi. His idea is, apparently, that it is merely a question of determining either the proper slope, angle, curve, or some other function of the form. As a matter of history, the river engineers in this country have seen these spur-dikes used elsewhere and have tried them extensively at home. Speaking in very general terms, the writer would say that when depths are relatively shallow spur-dikes may be used for contraction purposes, but, in deep caving bends, they are a total failure—as one engineer recently expressed it, "experience has shown that the construction of spur-dikes is the way not to protect a caving bank."

Mr. Freeman has referred† to the subject of scouring during flood and of bank protection, but he fails to explain what character of material will be used in order that the model bank, 1 or 2 ft. high, may either slip and slide in a manner similar to those of clay, 100 ft. high, along the Mississippi, or may break off in blocks, as happens with the sand banks along the real river.

Mr. Freeman has referred† to the subject of scouring during flood and refilling as the flood subsides. At the lower end of the Passes, there is normally a heavy deposit of soft mud during the low-water season, due probably to the fact that salt water at such times penetrates the Passes, flocculates the suspended mud, and thus aids in its deposition. These deposits, sometimes more than 10 ft. deep, are scoured out during the early days of the annual floods. The cutting and filling just described is due to a special condition at the very mouth of the river. At the Head of Passes numerous surveys have shown that the high waters do not scour out a deep channel, nor do excessive deposits occur during the falling or low-water stages.

* *Proceedings*, Am. Soc. C. E., August, 1923, p. 1235.

† *Loc. cit.*, pp. 1227 and 1230.

Assuming for the sake of argument that an average cut of 2 ft. took place from Cairo to the Gulf over an average width of $\frac{1}{3}$ mile, the volume of sediment thrown into suspension would be equivalent to 1 sq. mile nearly 700 ft. deep. This is more than double the total quantity of sediment carried by the river during a normal flood as computed by Humphreys and Abbot. If Mr. Freeman will apply the same method of reasoning to the cases of the Yellow and Colorado Rivers, it is believed that he will be forced to the conclusion that deep scour during floods is a local phenomenon and cannot possibly extend over sections of a river several hundred miles in length.

"Training a river to dig deep" is an attractive sounding phrase, but the writer knows of no instance where such a result has been attained in practice under circumstances similar to those found along the Mississippi. For many years, the river at and below Baton Rouge has been called on to carry a considerably larger volume than was the case before the white man's levees stopped the overflow of both banks. The increased discharge has not materially changed that part of the cross-section below the level of the old banks, although flood heights, flood cross-sections, slopes, and velocities have all been increased. The removal of the Red River rafts lowered water levels above, just as the removal of a dam will drain a pond. The Atchafalaya is busily engaged in building a channel through the swamp where none formerly existed, but the discharge of this river where the channel building is in progress has been increased several hundred per cent. in the last 100 years. At the mouth of the Mississippi, the Passes extend their deep-water stems into the bars as the latter advance Gulfward. All the cases mentioned are highly specialized and are not universally applicable.

Mr. Freeman also states that the Yellow River carries from 8 to 10% of sediment during flood. Along the banks of the Mississippi, rice farmers flood their fields to an average depth of about 2.5 ft. each year. If this irrigating fluid contained 8% of solids by weight, or 5% by volume, the average depth of deposit would amount to about 0.125 ft. per year. At this rate, in a time that would look very short in the history of China, the fields would be raised many feet. The dredge pipes used by the Miami Conservancy District in constructing its mammoth dams are understood to have carried on an average about 8% of solids which further illustrates the consequences that must follow from such a condition. The writer does not conceive that the Yellow River carries any such load of sediment. If the fact is as stated, it follows that conditions along that river must be so different from those prevailing along the navigable rivers of the United States that methods of treatment must also be radically different. Mr. Freeman refers to this sedimentary load in the Yellow River as being "more than double the highest percentage reported distributed broadly in the Mississippi." As a matter of fact, the heaviest load observed in the Mississippi at New Orleans for several years was about 0.27%, or about one-fifteenth the amount stated by the author as being distributed broadly in the Mississippi.

RAILROAD TRANSPORTATION AND RAILROAD TERMINALS

A SYMPOSIUM

Discussion*

By Messrs. B. F. Jakobsen and William T. Lyle.

B. F. Jakobsen,† M. Am. Soc. C. E. (by letter).‡—The railroad problem is not merely one of engineering; it is rather a problem in applied social and political economics. The railroads exist in order to serve the public and not *vice versa;* and railroad policy, therefore, must conform to the best interests of the public as decided by the public.

Railroading is a public business, because it affects and concerns every one whether or not he does business directly with the railroads. It is not a question of what the Government and the public has done for the railroads, as discussed on page 1443§ of the paper by Charles A. Morse, M. Am. Soc. C. E., entitled "Consolidation of Railroads". The Government and the public have done at least as much for the railroads as they have for the brewing and distillery interests. Yet, the public decided to abolish these industries, decided to declare them anti-social, and forbade them to operate. This is certainly a most fundamental right of a sovereign people. To regulate the railroads, the public, therefore, need not consider what it has or has not done for the railroads; as a matter of fact and of right, the public will consider what the railroads are doing and what they should do.

On page 1441§, Mr. Morse states that "the railroads of this country developed along rational lines up to the passage of the Sherman Act * * *." The "rational lines" to which he refers, must be the rebate frauds, stock manipulations, and gambling, indulged in by the various railroad "kings" and their confrères. Mr. Morse seems to forget entirely that the public was forced to regulate the railroad business in order to protect itself. As a remedy, he now proposes that all rate-making powers be taken from the Government, but he does not even attempt to show that those now in control of the railroads are any more competent, mentally and especially morally, than those whose conduct was responsible for regulation.

Mr. Morse repeats the often heard charge that Government operation of the railroads during the World War was an expensive failure; in fact, on page 1440§ he states that it was "even a greater disaster to the country than the

* Continued from October, 1923, *Proceedings.*

† Cons. Engr., Fresno, Calif.

‡ Received by the Secretary, September 24, 1923.

§ *Proceedings,* Am. Soc. C. E., September, 1923.

war itself * * *." The Government, however, took over the operation of the roads, because the railroad executives could not operate them. The executives state that this was not due to any fault or lack of ability on their part, but was due to Government interference, or labor union interference, or to both. The fact is that the executives failed to operate the roads, failed to accommodate their institutions to the social conditions, as it is their duty. Society is under no obligation to accommodate itself to railroad conditions, and it is known from experience that it will not do so.

On page 1447,* Mr. Morse suggests that the roads be allowed to handle the labor problem on the basis of supply and demand; that they be allowed to make their own rates, but with a minimum rate set by the Government—in other words, open and free competition, as far as labor is concerned, but a guaranteed minimum earning for capital, no matter how badly the management and engineering may be. The modern doctrine, however, is quite the reverse; out of the earnings of an industry, labor first takes its adequate share, and what is left goes to capital.

Mr. Henry Ford has recently shown what may be done with weak roads, in connection with the Ironton, Toledo, and Detroit Railroad. In his book, "My Life and Work," he states that legal expenses were reduced from $18 000 to $200 per month; the entire "administrative staff" was dispensed with; about 40% of those on the payroll were also eliminated, and a minimum wage of $6 per 8-hour day was placed in operation. In spite of this considerable reduction of force, Mr. Ford is able to handle more traffic than before and has placed the road on a paying basis.

On page 1446,* Mr. Morse states that "during 1922, the railroads paid more than $304 000 000 in taxes. * * * With Government ownership of railroads, there would be no taxes paid on the railroads * * *." This is a peculiar argument, inasmuch as the railroads did not "pay" these taxes or any part of them, but merely collected them from the patrons of the roads.

The attitude of Mr. Morse toward the public men of the nation should not be allowed to go unchallenged. On page 1440,* appears the following: "To-day, no one denies that regulation of great industries is desirable to prevent them from developing into monopolies, but Government officials and politicians are not content with regulation limited to that extent. The former crave power and the latter see the opportunity to extend their patronage and thus to extend their tenure of office * * *."

On page 1444,* Mr. Morse states that "the railroads are regulated in every way that the fertile mind of politicians can think of * * *"; and on page 1446,* in the discussion of what would happen if Government ownership should become a fact, he states: "New station buildings would come only through Congressional favor, and would be in the 'pork barrel' class."

From the writer's own limited experience he is well satisfied that the average public man, that is, the politician, is actuated by a profound desire to assist his fellow man; if this were not true, civilization would not be what it is to-day. The average politician stands morally on a very much higher plane

* *Proceedings*, Am. Soc. C. E., September, 1923.

than the average captain of industry, who seeks merely to enrich himself, and whose ideal is large profits with small wages. When a large number of our captains of industry are willing to serve the public, to accept the same small remuneration with which the Government official and the politician are satisfied, when they become capable of sharing the politician's interest in society and of looking on great statesmen, scientists, and artists, not as poor simpletons who, as benefactors of humanity, do for nothing that for which they might have exacted an enormous price—then the millennium will be at hand. Meanwhile, let us try to imitate our public men and not belittle and revile them.

In order to solve the railroad question acceptably to the public, it is necessary that the best engineering talent co-operate with the spokesmen of the public. If we fail in this, sentiment in favor of Government ownership will grow and as soon as it has become strong enough, no amount of argument will suffice to ward it off.

The history of the prohibition movement is well worth studying in this connection. If the liquor interests had accepted some rather moderate reform measures in time, the movement would no doubt have been stopped. If the railroads cannot reform from within on their own initiative, they will be taken over and operated by the Government, and not on any guaranteed earning basis. That which is left after all operating expenses are paid, will go to the owners. Even under such conditions they will fare much better than the owners of breweries and distilleries did when the prohibition amendment went into effect.

William T. Lyle,* Assoc. M. Am. Soc. C. E. (by letter).†—In the Symposium, one important aspect of the transportation and terminal problem has been omitted, namely, railroads and city planning. It is true that the relationship has been referred to many times, and in the paper entitled, "Street Development in Relation to Railroad Terminals",‡ by Jacob L. Crane, Assoc. M. Am. Soc. C. E., the relationship of railroad terminals to the city plan is briefly stated. If Mr. Crane's subject had been more comprehensive, an opportunity would have been afforded to develop this relationship.

As transportation is related to National development, so railroad terminals are related to the city plan. Railroads exist for the cities and are dependent on municipal prosperity for their successful operation. Although this must be evident, it is also true that cities cannot prosper while the railroads languish. Reverting to the railroad companies, it can be stated safely that, as business concerns, they cannot afford to continue the practice of strangling municipal growth. The relation between street development and railroad terminals is important, but more comprehensive and important is the subject which has not been treated, namely, railroad terminals as related to the city plan. Zoning, housing, parks, navigation, and the location of industrial plants are intimately related to the railroad problem in its terminal aspects. No terminal problem can be properly solved, unless it is considered as a part of

* Prof. of Civ. Eng., Washington and Lee Univ., Lexington, Va.

† Received by the Secretary, September 29, 1923.

‡ *Proceedings*, Am. Soc. C. E., September, 1923, p. 1519.

the comprehensive city plan; no adequate city plan can be prepared without a full and exhaustive study of the railroad problem. As the interests are mutual, evidently a joint commission is desirable. In his instructive paper, on "Modern Rail and Water Terminals,"* Major Rufus W. Putnam, Corps of Engineers, U. S. A., points out other interests of a National character which, for cities such as New York and Chicago, must be considered carefully. Railroad terminals are public as well as private problems the solution of which demands the benefits of a modern and rapidly advancing engineering art.

* *Proceedings*, Am. Soc. C. E., September, 1922, p. 1552.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

GEORGE RICHARDSON HARDY, M. Am. Soc. C. E.*

DIED APRIL 2, 1903.

George Richardson Hardy, the son of William Hardy, was born on June 9, 1848, at Hollis, N. H. When he was ten years of age, his family moved to Malden, Mass. He attended the Grammar and High Schools of Malden, and, in 1866, entered the Massachusetts Institute of Technology, where he studied Civil Engineering. He spent three summer vacations in engineering work connected with the office of the late J. Herbert Shedd, M. Am. Soc. C. E., in Boston, Mass.

In 1868, Mr. Hardy was engaged in making surveys for the establishment of street lines and grades for the central section of the City of Providence, R. I. In 1870, he was appointed Assistant Engineer on the Northern Pacific Railroad, on preliminary and construction work in Minnesota, and during 1871, he was engaged in mercantile work.

In 1872, Mr. Hardy entered the firm of Hardy and Kimball, Civil Engineers, with an office in Boston. In 1873, and for thirteen years following, he was employed by the Boston and Albany Railroad Company, first as Superintendent of Construction, in charge of a new elevator, shops, depots, etc., and afterward as Assistant Chief Engineer, in which capacity he had experience in maintenance of way, as well as in the design and construction of bridges, buildings, signals, and other structures. One of his most noteworthy accomplishments was the design and construction of the yards and track approaches to the Union Station, at Worcester, Mass.

In 1886, Mr. Hardy went West and was engaged as Assistant Chief Engineer, with the Lake Shore and Michigan Central Railroad Company, of which road he was appointed Chief Engineer in 1887.

From December, 1887, to August, 1889, he was with the New York, New Haven and Hartford Railroad Company as Assistant Engineer on Surveys. He was afterward employed by the Westinghouse Electric and Manufacturing Company, and as he was much interested in the introduction of the interlocking system of switches and signals in New England, he prepared and read an interesting paper on the subject before the Association of Engineering Societies in 1884.†

Mr. Hardy served as Assistant Engineer of Construction with the New York, New Haven and Hartford Railroad Company from January, 1893, to September, 1897, in which capacity he was engaged on special work, including the supervision of the 4-track construction on the New York Division, through the City of Stamford, Conn., and for several miles on each side of that city.

* Memoir compiled by Frederic I. Winslow, M. Am. Soc. C. E.

† "Improved Signal Apparatus Used on the Boston and Albany Railroad", *Journal*, Assoc. of Eng. Societies, Vol. IV (1884), p. 35.

From September, 1897, until July, 1899, he had immediate supervision of the elimination of grade crossings in Suffolk and Norfolk Counties, Massachusetts, and from July, 1899, until his death, he supervised the grade-crossing work at Blackstone, Whitins, and Readville, Mass. This included the construction of the extensive shops and tracks at Readville, also preliminary work pertaining to the proposed elimination of grade crossings in Worcester, Attleboro, Pawtucket, and Taunton, Mass.

Mr. Hardy was an excellent mathematician and often worked on intricate mathematical problems in search of simple solutions. He was of a cheerful and kindly disposition, dignified and gentlemanly in manner, and had very high ideals. Of a social nature, he had the happy faculty of making and keeping his friends.

He was elected Treasurer of the International Roadmaster's Association, which was organized in Boston on March 25, 1879, and was the first organization of its kind in the United States. He was also a member of the Boston Society of Civil Engineers and the Union League of New Haven, Conn.

He was married to Miss Ella C. Foljambe, daughter of Samuel W. Foljambe, D. D., of Malden, Mass. They had four children, three daughters and one son.

Mr. Hardy was elected a Member of the American Society of Civil Engineers, on November 7, 1888.

LOUIS HENRY KNAPP, M. Am. Soc. C. E.*

DIED JANUARY 16, 1923.

Louis Henry Knapp, the second son of Stephen Lockwood and Hannah (Bates) Knapp, was born on November 17, 1848, in Buffalo, N. Y. His ancestors were among the first settlers of Plymouth, Mass.

He attended the public schools of Buffalo and was graduated from the Central High School in 1865, at the end of his Junior Year, at the same time completing two years of medical work. In the fall of 1865, he entered Union College, Schenectady, N. Y., as a Sophomore in the Engineering Department and, in 1869, after three years of successful work, he was graduated as a Civil Engineer, one of the first three in his class.

At this time, and for many years after, a regular Army officer was detailed at Union College in charge of military training, and it was the custom to offer commissions to the most promising students who had taken the Engineering Course. Thus, following his graduation, Mr. Knapp entered the United States Army and was assigned to duty at Fort Lincoln, Nebr. After some months of active service, however, he was honorably discharged. After leaving the Army, he entered the service of the New York National Guard. He was made a Major of the Engineer Department of the 14th Brigade, 8th Division, on April 16, 1881, and Lieutenant Colonel of the 4th Division on June 3, 1882.

* Memoir prepared by Emile Low, M. Am. Soc. C. E.

On November 18, 1912, after thirty-two years of service, he was placed on the retired list with the rank of Colonel.

Mr. Knapp's first engineering work was with the Boston and Albany Railroad as Superintendent of Construction. He was the first to use four locomotives together to clear the track of snow, which, in those days, was quite an accomplishment.

In 1870, he was appointed an Assistant Engineer of the City of Buffalo, in charge for the Water Department of the tunnel and inlet pier under the Niagara River. This tunnel and inlet pier were the first thus built, and were completed in 1875. He held this position until July 1, 1873, when he was appointed Chief Engineer of the Department and of the Water-Works Improvement of the City of Buffalo, continuing as such until 1876, when he was appointed Engineer of the Water-Works of Niagara Falls and Suspension Bridge, N. Y.

From 1877 to 1878, he was in charge of the construction of the new reservoir and water-works at St. Catharines, Ont., Canada, which he completed satisfactorily, in spite of the sentiment in the community against having an American in charge of the work. At this time, Mr. Knapp met Lord Strathcona who was building the Canadian Pacific Railroad, and as soon as the work at St. Catharines was finished, he undertook, at Lord Strathcona's request, the work of surveying 500 miles of this railroad in the Canadian Northwest. The road traversed the country inhabited by the Blackfoot Indians, and Mr. Knapp was called by them the "Chief with the Glasses". He was given twelve eagle feathers by their Chief for bravery and, later, they sent him a box of furs in appreciation of his fair dealing with them. When the survey work was finished, the funds of the Canadian Pacific Company being greatly depleted, Mr. Knapp accepted his salary in railroad stock.

From 1879 to 1882, he served as City Engineer of Buffalo, and, in 1882, he was appointed by the late President Cleveland, then Mayor of Buffalo, as Superintendent and Chief Engineer of the Water-Works of Buffalo, which position he held for twenty-eight years. During this time, two more tunnels were constructed under the Niagara River. The last tunnel, which was built by the compressed air method, had a capacity of 7 000 gal. per min., and was finished on November 28, 1896. The present storage and distributing reservoir, which was designed by the late William Jarvis McAlpine, Past-President, Am. Soc. C. E., and constructed by Mr. Knapp, with Mr. John Record as Assistant Engineer, was built on the old Dodge Farm on the "East Side" between Dodge, Best, Masten, and Jefferson Streets."* Water was turned into its West Basin on November 16, 1893, and into its East Basin on July 6, 1894. The Buffalo hydrant was designed by Mr. Knapp in 1882 and was extensively used.

During these years, the old water pipes were relaid and, in 1896, a 48-in. pipe was laid through North Street from the pumping station to the new reservoir. It was the first 48-in. pipe laid in the United States and was a com-

* Described and illustrated in *Engineering News*, January 10, 1891.

plete success. Engineers came from all parts of the country to examine it and to be assured that the joints could be made tight.

In 1904, Mr. Knapp entered the competitive examination of engineers for the position of Resident Engineer on the New York State Barge Canal, and passed seventh on the eligible list. His thesis was on the design and construction of locks and other structures on the proposed canal. In 1904, Mr. Knapp also constructed a 5-span bridge over the Alleghany River at Onoville, N. Y.

From 1905 to 1920, he was engaged as Consulting Engineer on hydraulic projects, principally in confidential examinations, and also reported on electric railroads, water plants, and railroads for bondholders, his reports including first cost, principal values, revenues, and expenditures.

He was a member of the New England Water-Works Association. He was a Thirty-second Degree Mason and belonged to the Hugh de Payens Commandery, No. 30, Knights Templar. He was also a member of the Sons of the American Revolution.

On November 10, 1910, Mr. Knapp was made a Trustee of the Buffalo City Cemetery and until his death was active in its interests. He drew the plans for the Chapel and supervised its erection. He also beautified the course of the Scajaquada Creek, a natural waterway through the cemetery. Practically his entire life was spent in the service of the City of Buffalo, N. Y.

On May 30, 1883, Mr. Knapp was married to Miss Elizabeth Bruce Williams, of New York City, who, with a daughter, Ethelind, survives him.

Mr. Knapp was elected a Member of the American Society of Civil Engineers on March 4, 1874.

WILLIAM PARKER, M. Am. Soc. C. E.*

Died September 30, 1909.

William Parker was born at East Bridgewater, Mass., on August 26, 1861, and received his education at the local High School. In 1880, he accepted his first position as Rodman with Thomas Keith, a Land Surveyor of Brockton, Mass. From September, 1881, to January, 1882, he was employed as Rodman on 75 miles of preliminary railroad surveys in Connecticut and Massachusetts.

In 1882, Mr. Parker entered the service of the New York and New England Railroad Company, with which he remained until August, 1884. During this period, he acted as Rodman on double-track and miscellaneous work, and, from April to August, 1884, as Resident Engineer on double-track work.

Mr. Parker was employed for six months in the office of the late Alexis H. French, M. Am. Soc. C. E., Town Engineer of Brookline, Mass., as Sewer Inspector and Assistant in charge of private street construction. From March, 1885, to March, 1887, he served as Assistant in the office of the Division Engineer of the Boston and Albany Railroad Company, at Springfield, Mass., on general office and field work, and from March to November, 1887, he was engaged as Assistant Division Roadmaster on the Worcester to Springfield

* Memoir compiled by Frederic I. Winslow, M. Am. Soc. C. E.

Division of the same road. He was afterward made Principal Assistant Engineer which position he held at the time of his death.

There is little reason to doubt that the comparatively early death of Mr. Parker was due to his absorbing interest in his work as an engineer, for while others were enjoying summer evenings in relaxation, the light would burn late in his study. While his vitality was being lowered, he fell a victim to typhoid fever, from which he lacked the strength to rally.

In 1885, he was married to Miss Abbie A. Stoddard, of Douglas, Mass.

Mr. Parker's success in life was due to his own efforts; he had a remarkable faculty of observation, and was a most agreeable companion. He was an example of the highest type of manhood, and left behind him an untarnished name. His judgment as an engineer was excellent, and he was an exact and conscientious worker.

He was a member of the Boston Society of Civil Engineers. In 1898, he wrote an interesting paper on the "Abolition of the Grade Crossings on the Main Line of the Boston and Albany Railroad, in Newton, Mass."*

Mr. Parker was elected a Member of the American Society of Civil Engineers on March 7, 1900.

FRANKLIN COGSWELL PRINDLE, M. Am. Soc. C. E.†

Died March 6, 1923

Franklin Cogswell Prindle, the son of Hawley and Olive (Andrew) Prindle, was born on July 8, 1841, at West Sandgate, Vt., his ancestors on both sides having served in the Revolutionary and Colonial Wars.

He received his early education in the public schools of West Sandgate and Arlington, Vt., and always exhibited a great fondness for reading and study. His progress in mathematics was such that the Superintendent of Public Schools mentioned him in his official report.

Possessing a notable aptitude for mechanics and machinery, and having a strong dislike for farm work, his father's calling, he became an apprentice in the machine shop and foundry of Grover and Harrington, in Bennington, Vt. On account of his health, however, he was compelled to abandon this congenial work and return to the farm.

Making use of his knowledge of tools and his experience in the machine shop, he fitted up an outbuilding as a workshop of his own, where he spent much of his spare time, often working late into the night on devices and machines planned to lessen the drudgery on the farm. He constructed in wood, a working model of a duplex steam engine in which the slide valve on one cylinder was actuated by the piston rod of the other in practically the same manner as that which obtains in the duplex steam pumps of to-day.

This work and several other inventions brought Mr. Prindle into prominence in the vicinity, and resulted in his securing employment in a steam

* *Journal* of the Assoc. of Eng. Societies, Vol. 21 (1898), p. 50.

† Memoir prepared from information on file at the Headquarters of the Society.

saw and stave mill then being built in a neighboring town. He assisted in erecting the engine, boilers, and other machinery of this mill, and remained there for a time, first as engineer and fireman, and eventually as Superintendent when he was only eighteen years of age.

In 1860, he left the mill and entered the Sophomore Class at Rensselaer Polytechnic Institute, Troy, N. Y. At the outbreak of the Civil War, in 1861, he entered the Engineering Corps of the Navy. His disability, due to his being under age, was readily removed by Secretary Welles, and he passed third in the competitive examination, receiving, on August 3, 1861, an appointment as Third Assistant Engineer.

Mr. Prindle was immediately ordered to duty on the U. S. S. *Ottawa,* one of the "ninety-day gunboats", which joined Commodore Dupont's squadron at Hampton Roads, Va., and with which he served in the battle of Port Royal, on November 7, 1861. Following this, he saw much active service with the South Atlantic Blockading Squadron. On April 21, 1863, he was promoted to Second Assistant Engineer, and a year later, he was ordered to special duty at the Novelty Iron Works, New York City, where he remained until the close of the war.

On September 11, 1865, Mr. Prindle resigned from the Navy, and entered the office of Norman W. Wheeler, Mechanical Engineer, New York City. Two years later, he received the appointment of Assistant Civil Engineer at the New York Navy Yard, and, after a short time, was transferred to the Navy Yard at Philadelphia, Pa., where he was placed in charge of public works and improvements. He was commissioned a Civil Engineer in the Navy on April 17, 1869, and continued on duty at the old Philadelphia Navy Yard. Later, he was assigned to the new League Island Navy Yard, and, as its first Civil Engineer, made the plans for its development, and designed and constructed the buildings and other improvements.

Mr. Prindle resigned on January 1, 1876, to return to private practice in Philadelphia, but, in 1879, he was prevailed upon to re-enter the Naval Service and accepted his former position, with the purpose of making it his life work. On July 22, 1879, he was again commissioned a Civil Engineer, U. S. Navy, and ordered to duty at the New York Navy Yard.

He subsequently served as Civil Engineer at the Navy Yards and Stations at Portsmouth, N. H., Boston, Mass., Newport, R. I., Brooklyn, N. Y., League Island, Pa., Norfolk, Va., Port Royal, S. C., Key West and Pensacola, Fla., Mare Island and Yerba Buena Island, Calif., Puget Sound, Wash., and Honolulu, Hawaii. The construction of the Naval Training Station at Yerba Buena Island in 1898-1900 was his last important work. His official duties on the active list of the Navy ended at the Naval Station in Honolulu in 1900. From there, he was invalided home and transferred to the retired list on February 27, 1901, with the rank of Rear-Admiral.

Admiral Prindle was intensely interested in the advancement of his Corps as an important branch of the Naval Service and in the securing of suitable recognition of its proper position. It is due largely to his earnest efforts that the Civil Engineer Corps was established as a permanent Staff Corps of the Navy.

During 1876-77, he was Engineer and Secretary of the American Dredging Company, Philadelphia, Pa., and in the fall of 1876, he visited England, Scotland, Belgium, Holland, and Germany to examine the systems of dredging and types of machinery used in those countries.

In 1889-90, while on leave of absence, he was employed as Engineer and Superintendent of the Carolina Oil and Creosote Company at Wilmington, N. C., the latter part of the time as its Secretary and Treasurer, with entire charge and management of the business. When the Aztec Oil Company, one of the pioneers of the famous Kern River oil field in California, was organized, he was elected a Director and Vice-President. After his retirement from active service in the Navy, he became President of the Company.

Admiral Prindle was a member of the Institution of Civil Engineers of Great Britain, the Franklin Institute, the Boston Society of Civil Engineers, the Technical Society of the Pacific Coast, and the University Clubs at Philadelphia, Boston, and San Francisco. He was also a member of the National Geographical and of the National Genealogical Societies, at Washington, D. C., and of several patriotic societies, having been a Companion of the Military Order of the Loyal Legion of the United States, of the Naval Order of the United States, of the Society of American Wars, and Compatriot of the Society of the Sons of the American Revolution.

He was Past Master and life member of Crescent Lodge No. 493, Free and Accepted Masons, as well as a life member of Temple Chapter No. 248, Royal Arch Masons, and of St. Albans Commandery No. 47, Knights Templar, all of Philadelphia, and a member of the Grand Lodge of Masons of Pennsylvania.

Politically, Admiral Prindle was a Lincoln Republican and in religion a Baptist. He affiliated himself with the activities of the churches of that faith in the various cities in which he was stationed, and took an earnest interest in their work.

He was married on May 19, 1864, to Miss Gertrude A. Stickle who died on September 15, 1876. Seven children were born to them. On September 25, 1878, he was married to Miss Sarah A. Cranston who died on April 22, 1892. On April 8, 1896, he was married to Mrs. Fidelia E. (White) Mead who survives him.

His death occurred at the Naval Hospital, Washington, D. C., on March 6, 1923, and he was buried with military honors on March 9, 1923, at Arlington National Cemetery.

Admiral Prindle was elected a Member of the American Society of Civil Engineers on March 4, 1874.

REUBEN SHIRREFFS, M. Am. Soc. C. E.*

Died August 31, 1904.

Reuben Shirreffs, who was of Scotch descent, was born at Liverpool, Nova Scotia, on May 26, 1852. His boyhood was spent in his native town where he

* Memoir compiled by Frederic I. Winslow, M. Am. Soc. C. E.

received his education. He had an excellent reputation at the High School as a boy of fine character and as a student above the average, especially in mathematics.

From May, 1872, to April, 1875, Mr. Shirreffs was a student in the office of Clemens Herschel, Past-President, Am. Soc. C. E., at Boston, Mass., and from the latter date until September, 1879, he was engaged with the Engineer Corps of the Sudbury River Aqueduct, at South Framingham, Mass., as a Draftsman.

The year following, Mr. Shirreffs went to Chicago, Ill., where he entered the employ of the Chicago, Burlington, and Quincy Railroad Company, as an Assistant Engineer, in charge of work on the Chicago Yard, with supervision of construction at the Western Avenue Roundhouse.

From September, 1880, to June, 1881, he was with the Holyoke Water Power Company, as Assistant Engineer. On leaving Holyoke in June, 1881, he went to Richmond, Va., where he became Engineer of Water Power, and, afterward, Engineer of the Richmond and Alleghany Railroad.

Following this engagement, as a member of the firm of Stewart, Shirreffs and Company, Mr. Shirreffs was engaged in building dams, bridges, and in other structural work. From March, 1887, to December, 1889, he was in charge of the construction of the New City Hall of Richmond, Va., and of rebuilding part of the Free Bridge from Richmond to Manchester.

In January, 1890, he became Second Assistant Engineer of the East Jersey Water Company, with headquarters at Paterson, N. J., and, for about five years, he was employed on the design and construction of long, steel, pipe conduits, and other works, for supplying water to Newark, N. J., and towns in the vicinity.

About this time, the late Frederic P. Stearns, Past-President, Am. Soc. C. E., was organizing an Engineering Corps for the design and construction of the Metropolitan Water-Works System of Boston, and, in 1895, he gave Mr. Shirreffs charge of the Designing and Drafting Department. While in this position, he had a large share in designing the Wachusett Dam, Reservoir, and Aqueduct, the Clinton Sewage Disposal Plant, and other parts of the Metropolitan Works.

Mr. Shirreffs resigned this position in February, 1899, and returned to Richmond to become Chief Engineer of the Virginian Electric Railway and Development Company, which position he held until July, 1902. His work consisted in designing, building, and equipping a large new steam and water power house, dam, and canal at the Falls of the James River.

As a large development at Great Falls, on the Potomac River, was contemplated, Mr. Shirreffs accepted the position of Chief Engineer with the Great Falls Water Power Company. Work on this project was suspended, however, in June, 1904. Soon after, Mr. Shirreffs suffered a nervous breakdown and, during a period when he was manifestly not himself, passed away at Washington, D. C.

Mr. Shirreffs was married in May, 1884, to Miss Edith Howard, of Richmond, Va., who died about 1890, and on October 15, 1902, he was married to Miss Emma Bruce, of Richmond. He was a man of great ability and high ambitions and was respected by all who knew him.

He was a member of the New England Water Works Association, and of the Boston Society of Civil Engineers.

Mr. Shirreffs was elected a Member of the American Society of Civil Engineers on June 4, 1890.

GEORGE EDWARD SLEEPER, M. Am. Soc. C. E.*

Died October 25, 1908.

George Edward Sleeper was born at Boston, Mass., on May 13, 1850, and received his early education in the public schools of that city.

At the age of twenty, Mr. Sleeper entered the office of J. G. Chase, City Engineer of Cambridge, Mass. After two years of work and study in this position, he became a Transitman on the Fitchburg Railroad, later accepting employment as Transitman with the Boston and Lowell Railroad, with which Company he remained until 1873.

From 1877 to 1879, he was employed at the United States Navy Yard, at Portsmouth, N. H., as Draftsman, and, from 1879 to 1885, he held the position of First Assistant Engineer of the Old Colony Railroad Company. He left this position to take charge of surveys, for the Massachusetts Drainage Commission, of Boston and vicinity, his work including the improvement of Stony Brook following an excessive flood on that water-shed. Mr. Sleeper was also connected for a short time with the Onset Bay Grove Street Railway in Massachusetts and, in 1887, he was appointed Chief Engineer of the Mohawk Water Company on reconnaissance for canals in Arizona. In 1888, he was engaged for a short time in private practice, which he relinquished to take charge of the new water supply for the Boston Navy Yard.

Mr. Sleeper then resumed his private practice and devoted most of his time to street railway construction, acting in the capacity of Supervising or Consulting Engineer to the following companies in New England, principally in Massachusetts, and in Canada: The New London Street Railway; Danbury Street Railway, and the Montville Street Railway, in Connecticut; Gloucester Street Railway; Rockport Street Railway (later, the Gloucester and Rockport Street Railway); Gloucester, Essex, and Beverly Street Railway; the reconstruction of the Citizen's Street Railway of Newburyport, the Georgetown, Rowley and Ipswich Street Railway; Worcester and Marlborough Street Railway; Milford, Holliston, and Framingham Street Railway; the street railways in Hopkinton and Medway; Milford and Uxbridge Street Railway; Westboro and Hopkinton Street Railway; Brockton, Bridgewater, and Taunton Street Railway; Brockton and East Bridgewater Street Railway; Bridgewater, Whitman, and Rockland Street Railway; New Bedford, Middleboro, and Brockton Street Railway; Gardner, Westminster, and Fitchburg Street Railway; White River Valley Railway; Pawtucket, R. I., Street Railway; Framingham, Southboro, and Marlborough Street Railway; Providence and Taunton Street Rail-

* Memoir compiled by Frederic I. Winslow, M. Am. Soc. C. E.

way; Boston and Worcester Street Railway; Ontario, Canada, Street Railway; and many others in Canada and New England.

Mr. Sleeper's active work covered a period of almost forty years, his later years having been spent principally on water-works and sewerage projects.

He was a man of a genial nature, impulsive and generous, and was well liked by his associates.

Mr. Sleeper was elected a Member of the American Society of Civil Engineers on February 3, 1904.

WILLIAM WATSON, M. Am. Soc. C. E.*

Died September 30, 1915.

William Watson was born in Nantucket, Mass., on January 19, 1834.

In 1857, he was graduated from the Lawrence Scientific School of Harvard University, receiving the degree of S. B. from the Engineering Department. He also won the Boyden Prize in Mathematics. On his graduation, he became a member of the Harvard Faculty as Instructor in Mathematics, and, in 1858, was appointed Instructor in Differential and Integral Calculus at the Lawrence Scientific School, where he remained until 1859.

Professor Watson then went abroad to study, and, in 1862, received the degree of Ph. D. from the University of Jena. He also took a partial course at the École Nationale des Ponts et Chaussées and then rejoined the Faculty of Harvard University. From 1865 to 1873, he was Professor of Mechanical Engineering at the Massachusetts Institute of Technology, and organized the first course in Engineering in that school. After 1873, he devoted his time to special work.

In 1873, he was United States Commissioner to the Vienna Exposition and wrote the Government report on Civil Engineering, Public Works, and Architecture.

In 1878, Professor Watson was a member of the International Jury at the Paris Exposition, and Honorary President of the Paris Congress of Architects. In addition, in the same year, he served as Vice-President of the International Congress of Hygiene. In 1878, '81, '83, and '89, he was Honorary President of the Engineering Section of the French Association for the Advancement of Science, and in 1889, he also served as Vice-President of the International Congress of Construction. During this time, Professor Watson prepared the Government report on Civil Engineering for the Paris International Exhibition. In 1893, he acted as Secretary of the World's Columbian Water Commerce Congress.

Professor Watson was a member of the French National Academy, the Cherbourg French Society of Civil Engineers, the American Society of Mechanical Engineers, and many other technical societies. He was a Fellow of the American Academy of Arts and Sciences of which he was Secretary for a number of years, and of the American Association for the Advancement

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of Science. Among the other organizations to which he belonged were the Colonial Society of Massachusetts and the St. Botolph, Athletic, Round Table, Mathematical, and Physical Clubs of Boston. He was also the author of many mathematical textbooks.

In 1873, he was married to Miss Margaret Fiske, of Boston, Mass.

Mr. Watson was elected an Associate of the American Society of Civil Engineers on March 1, 1882, and a Member on September 2, 1891.

RALPH ALEXANDER ROLLO, Assoc. M. Am. Soc. C. E.*

Died February 25, 1923.

Ralph Alexander Rollo was born in Streator, Ill., on September 22, 1882. When he was quite young, his parents moved to Murphysboro, Ill.

He obtained his early education in the public schools of his native town, and was graduated from the Murphysboro Township High School with honors, thereby winning a scholarship to the University of Illinois. While attending the State University, he became affiliated with the Triangle Fraternity.

During the summer vacations, Mr. Rollo was connected with various engineering enterprises. In the third year of his University work, he accepted a position as Leveler and Transitman on the East Side Levee and Sanitary District, at East St. Louis, Ill., and, in November of the same year, he became Transitman for the St. Louis and San Francisco Railroad. In March, 1910, he established an office for himself, engaging in private practice, principally mining, chiefly for various coal companies in Illinois, and municipal work.

In May, 1911, Mr. Rollo became City Engineer of Murphysboro, and designed and superintended the construction of brick, asphalt, concrete, and bitumen bound pavements and sidewalks, and sanitary and storm sewers. In 1921, he was appointed Consulting Engineer for the Cities of Herrin and Carbondale, Ill., and prepared and designed certain public improvements in those cities, aggregating approximately \$1 000 000, which have been and are now being finally constructed.

Mr. Rollo was a conscientious, industrious, and efficient engineer, as is memorialized by his various works. He was held in the highest esteem by all those with whom he came in contact. His greatest attribute, attested to by all the city officials and by the various contractors who had occasion to work with and under him, was the fact that he was always on the "level". His was a unique character, honest, sober, capable, he was strictly a family man, always a student, and, in his death, the community has lost a real man.

On November 24, 1910, he was married to Cora Richards Brown, of Cora City, Ill., who, with a daughter, Jane, survives him.

Mr. Rollo was elected an Associate Member of the American Society of Civil Engineers on September 10, 1918. He was also a member of the Illinois Society of Civil Engineers.

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* Memoir prepared by Messrs. David B. Levy and George R. Johnston, Murphysboro, Ill., and T. N. Jacob, M. Am. Soc. C. E.